MAT-88-03

Competent Scour Velocity of Cohesive Soils

Ontario

Ministry of Transportation

Research and Development Branch

Technical Report Documentation Page

Competent Scour Velocity of Cohesive Soils

Author(s):	J.W. Kamphuis, Dept. of Civil Engineering, Queen's University
Number:	MAT-88-03
Date of Publication:	December 1988
Published by:	Research and Development Branch, MTO
Ministry Contact:	J.J. Armstrong (416) 235- 5254
Abstract:	The scour resistance of cohesive soils was tested in the laboratory and related to soil parameters. It was found that both the initiation of scour and the scour rate were very much dependent upon the characteristics of any granular materials present in the eroding stream. Bridge crossings should be designed as if the whole cross-section consists of the material that armours the stream bed, rather than taking into account the properties of the cohesive soil.
Comments:	This work was funded under the Ontario Joint Transportation and Communications Program, Project Number 22336.
Key Words:	bridge crossings, cohesive soils, erosion, scour
Distribution:	Highway Design and Geotechnical Engineers
Copyright Status:	Crown copyright © 1988 Ministry of Transportation

MAT-88-03

Competent Scour Velocity of Cohesive Soils

J.W. Kamphuis
Department of Civil Engineering
Queen's University

Published by
The Research and Development Branch
Ontario Ministry of Transportation
Hon. Ed Fulton, Minister
D.G. Hobbs, Deputy Minister

Published without prejudice as to the application of the findings.
Crown copyright reserved; however, this document may be reproduced for non-commercial purposes with attribution to the Ministry.

For additional copies, contact:
The Editor, Technical Publications
Room 320, Central Building
1201 Wilson Avenue
Downsview, Ontario
Canada M3M 1J8

Telephone: (416) 235-3480
CNCP Dialcom Electronic Mail: 23:MIN006

December 1988

Table of Contents

1/ Introduction 1
2/ Earlier Work 2
3/ Final Scope of the Tests 4
4/ Note on Critical Shear Stress 5
5/ Note on the Competent Velocity Concept 6
6/ Technical Methods Used 7
7/ General Observations 10
7.1/ General 10
7.2/ Erosion Depth vs Shear Stress in Clear Water 10
7.3/ Erosion Depth vs Shear Stress Under Sand Suspension 11
8/ Detailed Discussion of Soil Parameters 12
8.1/ Soil Description 12
8.2/ Geotechnical Parameters 12
8.3/ Critical Shear Stress 14
8.3.1/ Clear Water 14
8.3.2/ Sand Suspension 15
8.4/ Erosion-Shear Relationship 15
8.4.1/ Clear Water 15
8.4.2/ Sand Suspension 15
9/ Conclusion 16
10/ Recommendation 18

References 19

Appendices
1/ Collins Creek 61
2/ Humber River 63
3/ Fairchild Creek 65
4/ Dingman Creek 67

List of Tables and Figures

Tables

1/ Competent Velocities for Cohesive Materials 21
2/ Critical Shear Stresses 21
3.1/Erosion-Shear Stress Relationship 22
3.2/Erosion-Shear Stress Relationship 23
4/ Summary of Soils Properties and Erodibility 24
5/ Mineralogical Composition 25

Figures

1/ High Speed Flume- Tunnel 26
2/ Soil Press 26
3/ Shields Diagram 27
4/ Measuring Templates 28
5/ Erosion Depth-Shear Stress 29
5a/ Erosion Depth-Shear Stress 30
6/ Erosion Depth-Shear Stress 31
7/ Erosion Depth-Shear Stress 32
8/ Erosion Depth-Shear Stress 33
9/ Erosion Depth-Shear Stress 34
10/ Erosion Depth-Shear Stress 35
11/ Erosion Depth-Shear Stress 36
12/ Erosion Depth-Shear Stress 37
13/ Erosion Depth-Shear Stress 38
14/ Erosion Depth-Shear Stress 39
15/ Erosion Depth-Shear Stress 40
16/ Erosion Depth-Shear Stress 41
17/ Erosion Depth-Shear Stress 42
18/ Preconsolidation Stress vs Original Water Content 43
19/ Soil Sample 44
20/ Soil Sample 45
21/ Critical Shear Stress vs Original Water Content 46
22/ Critical Shear Stress vs Final Water Content 47
23/ Critical Shear Stress vs Clay Content 48
24/ Critical Shear Stress vs Plasticity Index 49
25/ Critical Shear Stress vs Precon Stress 50
26/ Critical Shear Stress vs Shear Strength 51
27/ Clear Water Average Values 52
28/ Erosion-Shear Slope vs Original Water Content 53
29/ Erosion-Shear Slope vs Final Water Content 54
30/ Erosion-Shear Slope vs Clay Content 55
31/ Erosion-Shear Slope vs Plasticity Index 56
32/ Erosion-Shear Slope vs Precon Stress 57
33/ Erosion-Shear Slope vs Shear Strength 58
34/ Clear Water Average Values 59
35/ Clear Water Average Values 60

1/ Introduction

Little information is available for calculating the scour resistance of cohesive soils. While the erodibility of granular soils such as sand and gravel has been studied extensively, the competent velocity of cohesive soils is approximated from very descriptive soil classifications such as soft, firm, stiff, etc.

The objective of the present study was, therefore, to relate scour resistance of cohesive soils to quantifiable soil parameters which normally result from geotechnical investigations. These are soil strength, plasticity, liquid limit, type of clay, percent clay, silt. Although the initial scope of the problem was to determine the fluid shear stress which causes initiation of scour and to translate these results into "competent velocity", the scope was later extended to investigate scour depth and the time element of erosion.

2/ Earlier Work

Three earlier studies were done at Queen's University using the same equipment as the present study. The first set of experiments was reported in Kamphuis and Hall (1983). An extensive literature review was done by Hall (1981) and a series of 24 tests was performed on two types of consolidated clay taken from the bottom of the Mackenzie River at Norman Wells, N.W.T. The samples were reconstituted from the original clay dug with a clamshell from the bottom of the Mackenzie River and were consolidated using a large press (Figure 1). Subsequent to consolidation, they were tested in the high velocity flume-tunnel specially constructed for this experiment (Figure 2). The size of the samples tested in the flume was about 600 mm long x 150 mm wide x 100 mm deep; i.e., the samples were large compared to normal soils test samples. Velocity distributions were measured above the sample and shear stress on the bed was computed from these velocity measurements. Critical values of shear stress were determined when the sample began to erode substantially. These were related to geotechnical parameters such as percent clay, percent silt, percent sand, natural water content, plastic limit, liquid limit, plasticity index, and unconfined compressive strength. The main conclusions of this test series were:

a) Critical shear stress varies linearly with compressive strength of the soil.

b) Critical shear stress increases with sample clay content and plasticity index.

c) Critical shear stress increases with consolidation pressure and, for consolidation pressures greater than 200 kPa, the soil may be considered safe from erosion by "naturally occurring" flow conditions.

d) Once the critical shear stress for a soil is reached, erosion progresses immediately; i.e., there is little or no dependence on the previous time history of the flow.

e) Once the initial clay surface has been altered, erosion progresses at a more rapid rate due to an increase in hydrodynamic roughness at the soil surface.

It was noted that once the critical shear stress was exceeded, the clay surface eroded by pitting and flaking. The latter is defined as the removal of thin flat particles.

A second set of six tests described in Kamphuis (1983), on similar materials to those used in the above series of tests, introduced a small amount of sand in the eroding fluid for portions of the

tests. The sizes and amounts of abrasive material were optimized so that the material bounced along the bottom, abrading the clay as opposed to either being completely in suspension or forming a discontinuous sand layer which would cover and protect the clay. The main conclusions of these tests were:

a) A fluid containing sand will cause erosion of a consolidated cohesive bed at a much lower shear stress than if the fluid were clear.

b) The erosion rate decreased with increasing consolidation pressure.

The tests also demonstrated an interesting erosion pattern in which high spots on the sediment bed were sandblasted while low sections were protected somewhat by a thin sand layer which deposited itself in these sheltered zones. The whole erosion process with sand included in the eroding fluid was quite different from the pitting and flaking seen in the first set of tests and in the clear water portions of the present tests.

The third relevant set of tests was performed on lakeshore till formations found in the vicinity of Port Burwell, Ontario. In this study, published as part of the documentation for litigation, Kamphuis (1983a), all earlier conclusions were clearly corroborated for these radically different materials. Differences in erosion characteristics were found between the various types of local tills. In this set of tests, the samples were not reconstituted and pressed together as in the first two sets; they were actually obtained from the shoreline in question, using a metal-frame box-corer. The use of undisturbed samples was found to be necessary if absolute values were required for the erosion characteristics.

In light of these earlier tests, the original objectives and scope of this study were broadened to incorporate the apparently much more critical and common scenario of small amounts of abrasive (sand or gravel) present in the eroding stream.

3/ Final Scope of the Tests

Since it is impossible to generalize from a limited series of tests, it was initially proposed that three typical problem soils be studied:

1) Western Ontario Till (Thames Valley),
2) Leda Clay (Ottawa Valley),
3) Lacustrine Clay (New Liskeard),

and that the experimental method be set up using the western Ontario till, which is easiest to handle.

Subsequently (July 15, 1987 meeting, Minute 5), it was decided to narrow the scope to western Ontario tills and only if these were completely understood, to work on the Leda and lacustrine clays. However, suitable samples were difficult to obtain and, while suitable western Ontario samples were being located, it was decided to carry out an extensive series of tests on "Collins Creek" material (Appendix 1) to set up the exact methodology

Once samples from western Ontario were obtained and preliminary tests were performed, considerable variation was found in results between the several similar soils and even between samples of the same soil taken a few metres apart. At that point it was decided to test in-depth the erosion by clear water and by water containing suspended sand for materials found at four Ontario bridge crossing locations:

1) Collins Creek (near Kingston)
2) Humber River (in Toronto)
3) Fairchild Creek (near Brantford)
4) Dingman Creek (near London)

These tests would provide replication of a smaller number of sampling locations, leading to a deeper understanding of a smaller number of clays. A few tests of many different clays would lead to almost meaningless results because the incompletely tested variations at any one site would obscure the variations between sites. It was felt that to test a few sites would provide the most useful results with the available resources.

The soil at all these locations may be described as lacustrine clay and the details of the test results are described in Appendices 1, 2, 3 and 4.

4/ Note on Critical Shear Stress

It is generally understood that the initiation of bottom scour can be related to the shear stress exerted by the fluid on the bed. This concept is quite clear when the soil is cohesionless and it is normally expressed by the well-known Shields Curve for initiation of sediment motion, a version of which is shown in Figure 3. In this figure, the critical condition for initiation of sediment motion is the dark line. For convenience, grain size has been plotted on this diagram and so the critical mobility number Y_c may be read directly if the grain size is known. For example, for a 2 mm diameter material, $Y_c = 0.05$. From this number, the critical shear stress τ_c may be computed using:

$$\tau_c = Y_c (\rho_s - \rho) g D \qquad (1)$$

where: ρ_s is the sediment density

ρ is the fluid density

g is the gravitational acceleration

D is the particle size

Thus for sand (ρ_s = 2600 kg/m^3) and with fresh water as fluid (ρ = 1000 kg/m^3), the critical shear stress required to move 2 mm material along a flat bed may be computed from Equation 1 to be $\tau_c = 1.6$ Pa. For a constant average stream velocity, the local velocity and the shear stress near the bed will decrease once a scour hole has developed. Thus, once critical shear stress is exceeded, scour will continue to such a depth that the shear stress is decreased below its critical value, and then scour stops. Although scour rate is also related to actual shear stress, a tacit assumption normally made in the critical shear stress concept is that scour occurs almost instantaneously, i.e., a scour hole develops in a relatively short time.

With cohesive soils, such a concept of critical shear stress is seriously flawed. If a cohesive material is homogeneous and free of fractures, such as in Kamphuis and Hall (1983), it is possible to define a critical shear stress, but for naturally-occurring *in-situ* samples, fracturing and softening of the clay by swelling upon immersion, as well as the presence of lenses of cohesionless material, will greatly influence initiation of scour, local erosion depth and the rate of erosion of the material.

5/ Note on the Competent Velocity Concept

Although sediment movement and initiation of scour are directly related to the shear stress exerted by the fluid on the sediment, to describe the process more simply and to make it easier to apply for practicing engineers, the concept of "competent velocity" is often used. Competent velocity is defined as "that average stream velocity which is just 'competent' to move the material exposed at the scoured depth" (Neil 1972, p. 90). Competent velocity in a steady-uniform flow must therefore be a function of flow depth as well as critical shear stress.

The usual table for competent velocity for cohesive soils (Neil (1972), Table 4-1; MTC Drainage Manual, Chapter H, Chart H8-3) categorizes "soil scourability" as high, medium and low. Such a table is reproduced in Table 1. High scourability is empirically related to "very soft to soft" clays defined subjectively as soil easily penetrated several centimetres by thumb. Medium scourability is empirically related to "firm to stiff" clays, i.e. clays that are readily indented but penetrated by thumb only by great effort. Low scourability is related to "very stiff and hard" clays, defined subjectively as soils which can be indented by thumbnail (MTC Drainage Manual, Chapter H).

Neil (1972) attaches critical shear stresses to these subjective soil scourability classifications (Table 2). To calculate competent velocity in Table 1 from the shear stresses in Table 2, Neil assumes an average Manning's n value of 0.025 and derives the expression:

$$V = 7.5\, d^{1/6}\, \tau^{1/2} \qquad (2)$$

where: V is the competent velocity
d is the water depth
τ is the shear stress from Table 2.

Equation 2 is a very approximate relationship and it assumes steady-uniform flow (certainly not valid at bridge crossings where flow is contracted and backwater effect often exists).

6/ Technical Methods Used

To test the scour of a cohesive soil so that some of the soil variability found in nature is included in the tests requires relatively large blocks of undisturbed soil. The high-speed flume at the Queen's University Coastal Engineering Research Laboratory (Figure 2) is designed to accept a sample with maximum surface dimensions of about 600 mm length, by 150 mm width, by 100 mm depth. Rectangular, steel box-corers with maximum inside dimensions of 595 mm length, by 138 mm width, by 75 mm depth were constructed to collect the soil samples in the field. The sample, together with the box-corer, fit into the test section of the flume to ensure minimum disturbance of the soil sample.

To collect the samples at each site, any sediment overlying the soil of interest was removed and a horizontal plane was smoothed off. The box-corer, with its sharpened edge facing downward, was placed on this horizontal plane with its long axis parallel to the axis of the stream. The box-corer was then slowly forced into the ground and the soil surrounding the outer edge was carefully removed. When the entire box-corer was forced into the ground, so that several centimetres of sediment protruded above the top edge of the corer, the sample was cut loose from the underlying soil with a length of fine piano wire drawn along the length of the corer. Upon removal from its surroundings, each sample was wrapped in plastic wrap, taped to a plywood board to prevent the sample from falling out of the box-corer, labelled, and then sealed in paraffin wax to prevent moisture loss. Five samples were collected at Collins Creek, five at the Humber River, four at Fairchild Creek, and five at Dingman Creek.

A standard procedure for testing the soil blocks was followed in the laboratory. Each sample selected for testing was trimmed with a wire cutter to provide a smooth top surface. An initial water content measurement was made from the material trimmed from the surface. The sample was then placed in the flume and carefully leveled to ensure that the sample surface was flush with the flume floor. Surface profiles of the sample were collected using a point gauge, giving a distance below an arbitrary reference datum. The measurement grid used is shown in Figure 4. All subsequent calculations of the erosion volumes are made with reference to this initially measured surface.

To initiate erosion of the sample surface using clear water, the flow was increased in increments of 0.2 m/s until general surface erosion was observed to occur. Once this general erosion began, the water velocity was maintained for two hours to allow a measurable amount of erosion to occur, as well as to permit observation of the pattern of scour of the soil. At the end of the two-hour period

the flow was stopped and the surface profiles were measured again at the locations in Figure 4. The flow was then re-initiated and the sample surface was monitored visually to see if any further erosion occurred. If no appreciable erosion was noticed, the velocity was increased in 0.2 m/s increments until the erosion process began again and then was held constant for another two hours. The two hour test duration was chosen after initial tests indicated that to be a reasonable time for erosion under a constant condition to cease. The erosion process was continued until more than 10 mm of material had been removed from the sample surface. After collection of the final surface profiles, the sample was inverted in the flume and the opposite surface was tested in a similar fashion.

In a number of the tests, sand was introduced into the water as an abrading agent. This sand had a mean size of approximately 2 mm and a minimum size of 1.4 mm.

During each two-hour test the water velocity was continually monitored and velocity profiles in the lower 60 mm of flow were collected at two Pitot tube stations; one upstream of the sample and one downstream. These velocity profiles were used to determine the shear velocity at the bottom. Hall (1981) found the flow in the flume to be in the hydraulically smooth regime of turbulent flow and to exhibit a logarithmic distribution. As a result, the well-known theoretical expression for turbulent flow over a smooth wall was applied to the flow:

$$\frac{v}{v_*} = \left[\frac{1}{\kappa}\right] \ln \left[\frac{9\, y\, v_*}{\nu}\right] \qquad (3)$$

where: v = the velocity measured at a distance "y" above the bed,
v_* = the "shear velocity"
ν = kinematic viscosity
κ = von Karman constant = 0.4.

For each pair of values of v and y measured during the tests, the shear velocity, v_*, was calculated and then averaged over all the values of y to yield a mean value of shear velocity. This shear velocity was converted into bottom shear stress using:

$$\tau = \rho\, v_*^2 \qquad (4)$$

Upon completion of the flume tests, the soil sample was removed and prepared for subsequent soils tests. Immediately upon removal from the flume, subsamples were taken for water content

analysis to determine whether or not significant absorption of water had taken place. In addition to the water content tests, subsamples were taken for grain size analysis, Atterberg limits and consolidation tests. The results of these soil tests are included in each appendix.

To document the material erosion, a large number of individual measurements were made on each sample, as shown in Figure 4. Six lines of measurements ran parallel to the length of the sample spaced at 25 mm intervals, L_1 to L_6. Each line consisted of 23 individual measurement points also spaced 25 mm apart , R_1 to R_{23}. The height of the soil surface relative to an arbitrary datum was measured at each of these points at the end of each two-hour testing period. The difference between the initial surface and each subsequent surface was calculated, yielding a value of erosion depth at each point.

An estimate of the surface location was obtained using trapezoidal rule integration as:

$$S = \sum_{L=2}^{5} \left[R_1/2 + \sum_{i=2}^{22} R_i + R_{23}/2 \right] \tag{5}$$

and average erosion depth at any time was computed as the difference between the surface calculated at the particular time minus the surface at time zero.

$$E = (S - S_o)/88 \tag{6}$$

where: E is the average erosion depth (mm)
S is the surface defined by Equation 5
S_o is the original surface at the beginning of the test
88 is the effective number of measurements (R) involved in Equation 5.

In addition to the erosion depth and shear stress measurements, a visual record of the samples was collected. Each sample was photographed prior to the initiation of any testing and following the completion of each two hour test. A video record was also made of a number of the sample tests, showing both erosion with clear water and with sand suspension. The soil tests performed on the samples followed standard engineering procedures.

7/ General Observations

7.1/ General

The figures and data presented hereafter and in Appendices 1 to 4 demonstrate sufficient scatter that the following interpretations cannot be numerically precise. The test results are summarized in Tables 3 and 4.

7.2/ Erosion Depth vs Shear Stress in Clear Water

Examination of the test results for each sample shows that erosion depth increases linearly with the shear stress exerted on the sample surface. This is demonstrated in Figures 5 to 8 which incorporate data from Table 3. For some of the larger erosion depths, there is some tendency for the points to become asymptotic to horizontal. This simply reflects the error in the shear stress calculation owing to sheltering. The plotted shear stress, based on a smooth law of the wall shear relationship (Equation 4), is no longer correct and shear stress in areas of substantial erosion depth is less than the values calculated.

The linear relationship of Figures 5 to 8 may be represented by:

$$E = m(\tau - \tau_c) \qquad (7)$$

where: E is erosion depth (mm)
τ is the shear stress exerted by the fluid on the sample (Pa)
τ_c is the critical shear stress (Pa)
m is the Erosion-Shear slope of the line (mm/Pa)

Values of critical shear stress and Erosion-Shear slope may be obtained directly from Figures 5 to 8 by drawing straight lines through the points as illustrated in Figure 5a. It is in this manner that the values of τ_c and m were determined for Table 4. For Figures 5 to 8, the Erosion-Shear slope (m) is seen to vary over a fairly narrow range from 0.75 to 4.3 with a mean value of about 2.0. The value of the critical shear stress τ_c, however, varies widely from 0 to 6. Both τ_c and m appear not to be very dependent on detailed soil parameters tested. For these inhomogeneous soils, the presence of silt and sand lenses in the sample and the extent of fracturing seem to determine the erosion characteristics.

7.3/ Erosion Depth vs Shear Stress under Sand Suspension

Figures 9 to 12 show that erosion by a sand suspension also varies linearly with shear stress, i.e., Equation 7 is also valid. Comparison with the clear water results, such as in Figures 13 to 16, indicates that τ_c now varies over a much narrower range from 0.6 to 1.9 for the sand suspension tests and that the Erosion-Shear slope (m) is consistently greater. An average value of m of 8.4 was found for the sand suspension tests, which suggests that the suspended sand is an active eroding agent on the cohesive samples as expected from Kamphuis (1983). Thus, at the same fluid shear stress exerted on the sample, the scour with sand is substantially greater but the flow velocities must be large enough to prevent the sand from forming a covering layer which actually protects the underlying cohesive material (e.g., Collins Creek Test 4T, Figures 9, 13 and 17).

The sand used in these tests had a mean diameter of 2 mm and a minimum diameter of 1.4 mm. Using the Shields diagram (Figure 3) and Equation 1, the critical shear stresses to move this sand may be computed to be 1.6 Pa for the 2 mm material and 0.9 Pa for the 1.4 mm sand. The sand used in these tests will, therefore, become mobile at a shear stress of 0.9 to 1.6 Pa. At that time, the sand cannot provide protection and begins to act as an abrasive, commencing the relatively rapid erosion of the cohesive sediment. Figures 9 to 17 substantiate this hypothesis, suggesting that the controlling influence in erosion of cohesive soil by fluid containing sand is not the cohesive soil or its properties but the properties of the abrading sand.

8/ Detailed Discussion of Soil Parameters

8.1/ Soil Description

The clays from Collins Creek, Dingman Creek, Fairchild Creek and Humber River are all quite similar. They are classified as CL (silty clay of low plasticity). The detailed descriptions of their sampling and *in-situ* condition are given in Appendices 1 to 4. All clays appear to be post-glacial and may be described generally as lacustrine clays. They all experienced some desiccation.

8.2/ Geotechnical Parameters

Several test blocks were taken from each site and the following geotechnical parameters were obtained for each block:

- % gravel sizes
- % sand sizes
- % silt sizes
- % clay sizes
- Liquid limit
- Plastic limit
- Plasticity index
- Original water content
- Water content after flume test
- Preconsolidation stress
- Undrained shear strength (= 1/2 x unconfined compression)
- Mineralogical composition

The values of the geotechnical parameters are given in Table 4, and the mineralogical composition in Table 5.

The soil classification data from the blocks within each site were consistent and were also fairly similar across all four sites. The percentage gravel sizes ranged from 0 to 5, percentage sand sizes from 0 to 19, percentage silt sizes from 42 to 76, and percentage clay sizes from 19 to 45. Liquid limits ranged from 27 to 42%, plastic limits from 16 to 22%, and plasticity indices from 5 to 23%. Original water contents were quite close for samples from the same site, ranging from 14 to 19% for Dingman Creek, and from 25 to 29% for the other three sites. Contact with water during the

flume test generally caused an increase in water content up to 10%. This increase is evidence of some desiccation and would lead to some swelling and softening of the clay.

The mineralogical composition of these clays is quite typical of those formed on Canadian Shield sedimentary materials. The clay mineral contents ranged from 29% to 46% and are all of low activity. No significantly different behaviours would be expected to result from the slight differences in mineralogical composition between these clays.

Preconsolidation stresses were generally in the 100 to 200 kPa range with some higher values, particularly for Dingman Creek with values from 260 to 300 kPa, and one Collins Creek sample at 400 kPa. These preconsolidation stresses are thought to be due to desiccation. The blocks were obtained mostly from exposed deposits within the crust where desiccation commonly has occurred. There was no relationship between preconsolidation stress and original water content (Figure 18). This is to be expected since preconsolidation stress is a result of past desiccation which is not necessarily reflected in present *in-situ* water content.

The clays contained many fissures, probably also a result of desiccation.

Undrained shear strengths were variable, with one Collins Creek sample at 160 kPa, Dingman Creek ranging from 58 to 212 kPa, Fairchild Creek from 70 to 139 kPa, and Humber River from 54 to 88 kPa. The variation in undrained shear strength reflects the effect of the fissures. Examples of these fissures can be seen in Figures 19 and 20, showing samples from Humber River and Dingman Creek, respectively, after unconfined compression testing and oven-drying. The Dingman Creek samples had the lowest original water contents and the highest undrained shear strengths, reflecting the effect of water content on shear strength.

As the blocks were taken from natural deposits, some variation in properties between blocks from the same site would be normal. The variations in plasticity and grain size distribution fell within a normal range. Because of this natural variation, the results for all the samples from each site were averaged. These average values, as well as individual values, were used to investigate any trends.

8.3/ Critical Shear Stress

8.3.1/ Clear Water

Observations of erosion behaviour in the flume tests indicated that erosion was initiated in the fissures at the clay surface, and also that flakes of soil were removed above thin sand and silt seams. Erosion behaviour was thus influenced by the presence of these local irregularities, and the values of critical shear stress were quite variable. For example, in some blocks where both top and bottom faces were exposed to clear water erosion, quite different values of critical shear stress were measured; for example:

H1T - 3.2 Pa
H1B - 0.7 Pa
H3T - 1.0 Pa
H3B - 4.0 Pa

while in one block, very similar values were found; for example:

C2T - 0.8 Pa
C2B - 0.7 Pa

There were no apparent relationships between the critical shear stress and any geotechnical parameters for individual samples (Figures 21 to 26). This is probably the result of the effect of the fissures overriding any other effects.

The average critical shear stress for each site is plotted against the average preconsolidation stress in Figure 27, indicating that the critical shear stress decreased as the preconsolidation stress increased. This can be explained as follows. The preconsolidation stresses are due to desiccation. A greater amount of desiccation resulted in not only higher preconsolidation stresses but also more fissures. The greater number of fissures resulted in initiation of erosion at a lower shear stress.

There was some indication from the Dingman Creek samples that the blocks with a higher sand and gravel content had a lower critical shear stress. This could be the influence of small sand seams leading to easier erosion, as suggested by Chapuis (1986).

Overall, it appears that the critical shear stress for these clays is influenced by fissures and sand seams rather than by any other geotechnical parameters. This conclusion contradicts the earlier findings of Kamphuis and Hall (1983) for a homogeneous, reconstituted clay where the critical

shear stress was found to increase with increase in shear strength and preconsolidation stress. It does, however, agree with the results of Lefebvre et al (1985, 1986) who concluded that for undisturbed sensitive clays, the critical shear stress was strongly influenced by the structure and fabric of the clay, i.e., the presence of fissures and small sand seams. They found there was no relationship between geotechnical parameters and critical shear stress for these undisturbed sensitive clays. Chapuis (1986) also referred to the importance of fissures on the erosion of clays.

8.3.2/ Sand Suspension

No relationship between the critical shear stress and any geotechnical parameters for the tests with sand suspension was observed (Figures 21 to 26).

8.4/ Erosion-Shear Relationship

8.4.1/ Clear Water

No relationship between the erosion-shear stress slope and the geotechnical parameters was observed for individual samples (Figures 28-33).

However, comparing sites using averaged values, there was perhaps some indication that the average rate of erosion decreased as both the average undrained shear strength and preconsolidation stress increased (Figures 34 and 35). This result appears to be reasonable as it would be expected that the more strongly the soil particles were bound together, the more difficult it would be to erode the soil. The presence of fissures would lead to earlier erosion but, once erosion had begun, the rate of erosion would be affected by the strength with which the soil particles were bound together. The scatter in the results due to the presence of fissures and sand seams precludes any more detailed conclusions.

8.4.2/ Sand Suspension

The Erosion-Shear relationship for the sand suspension tests were similar for each site and were much higher than those for the clear water tests (Figures 28 to 33). Correlation with geotechnical parameters was not possible.

9/ Conclusions

The present tests have shown:

a) Erosion depth increases linearly with excess fluid shear stress, i.e., with shear stress above a critical value.
b) Erosion depth is much greater for the same fluid shear stress with sand present in the water.
c) The critical shear stress when clear water begins to erode cohesive material, is very much a function of fracturing and swelling and cannot be readily defined from the usual soils parameters.
d) Erosion, with sand present in the water, begins when the sand becomes mobile and stops when the sand stops moving and forms a protective layer.

In the unusual case that the stream carries absolutely no granular, non-cohesive material, the designer needs to consider not so much the standard soils parameters such as soil strength, plasticity, etc., but more so the environmentally imposed conditions such as fracturing and swelling. The designer also needs to be aware of the presence of any non-cohesive elements, e.g., silt and sand lenses and layers which substantially change the erosion characteristics of the material. Finally, it was seen from these tests that the soil is normally inhomogeneous and that samples taken in close proximity of each other from the same stratigraphic unit can result in quite different erosion depths while subjected to the same fluid shear stresses.

For the more usual case in which granular material eroded from the cohesive soil or from neighbouring non-cohesive soil units is transported across the cohesive material surface by the water, it is clear that the stability of the coarser fractions of the eroded non-cohesive material determine the erosion of the cohesive sediment. As long as the fluid shear stresses remain sufficient to transport the granular material, it will sandblast and remove the cohesive material at a much higher rate than the clear water could under the same shear stresses. Once the shear stresses can no longer move the granular material, the sand or gravel will form a protective layer and the erosion of the cohesive sediment stops.

In either case, the present design philosophy which attempts to use soil classification parameters for the local cohesive sediment is wrong, regardless of how accurately the soils parameters are defined.

From the present tests, it is also clear that the erosion of the sediments tested occurs rather rapidly and thus it is possible to neglect any time element in the erosion process. Almost all tests reached stability after two hours, i.e., no further erosion occurred after two hours as a result of a substantial velocity increase of 0.2 m/s. The same was not true for Kamphuis (1983a) for the hard glacial tills recovered from the Port Burwell area. Those tills took some time to erode. Thus, unless the cohesive material is very hard, it is possible to neglect the effect of time.

The net outcome of the present tests is, therefore, to result in a design philosophy which, except for a very unusual case of complete absence of granular material in the stream, remains identical to that used for granular soils. The basic design problem remains similar to that for granular materials, i.e., to determine a stable grain size — the grain size that armours the bottom of the stream. Once that grain size has been estimated, the same design principles apply, i.e., scour occurs until the fluid shear stress is no longer sufficient to move this armouring material. At that time, the armouring material will protect the underlying soil — be it finer cohesionless material or cohesive material.

10/ Recommendations

Based on the extensive tests performed by Kamphuis and Hall (1985), Kamphuis (1983 and 1983a), and on the present tests, it is apparent that erodibility of a naturally-occurring cohesive material by clear water cannot be readily related to standard soils parameters. If granular material such as sand or gravel is present in the eroding stream, erodibility becomes very much a function of the granular material and the properties of the cohesive material only appear to play a secondary role.

Since erosion by a perfectly clear fluid would be an unusual situation and since assuming the presence of a suspended granular material errs on the safe side, the following recommendations are made:

1/ For soils similar to those tested, i.e., lacustrine clays found in southern Ontario, the design should be based on the mobility of unconsolidated armouring material carried by the stream and not on any of the properties of the cohesive material. This makes the problem of scour in cohesive soils very similar to that of scour in granular soils and rejects the present design philosophy based on soil strength parameters describing the cohesive soil.

2/ The same design philosophy should apply to leda clays and lacustrine clays found elsewhere, e.g., New Liskeard.

3/ For western Ontario tills, the same design philosophy should apply except that the rate of erosion is slower and hence a time element may enter the calculation. To this end, it is recommended that individual soil samples be tested for these tills, when required, to determine erosion rates.

4/ The presence of silt and clay in the water should be ignored; this is slightly conservative.

References

Chapuis, R.P. (1986), "Quantitative Measurement of the Scour Resistance of Natural Solid Clays", *Canadian Geotechnical Journal*,Vol. 23, No. 2, pp. 132-141.

Gillespie, J.E., R.E. Wicklund, and B.C. Matthews, (1966),"The Soils of Frontenac County". Canada Department of Agriculture, Ontario Soil Survey, Report 39, 67 pp.

Golder Associates, (1987),"Geotechnical Investigation, Proposed Bridge Replacement, Victoria Road, Concession III, Township of Westminster, County of Middlesex, Ontario". Report to Huron-Middlesex Engineering Limited, 11 pp.

Hall, R. (1981), "Initiation of Erosion of Consolidated Clays by Unidirectional Flow," M.Sc. Thesis, Queen's University.

Harris, J.D., "Design Flood Estimation for Medium and Large Watersheds," Chapter H. MTC *Drainage Manual,* February 1986.

Hunting Technical and Exploration Services Limited (1958), "Preliminary Site Investigation for the Proposed Collins Creek Bridge on Highway #401 near Kingston, Ontario". Report prepared for Ontario Department of Highways, 26 pp.

Johnson, C.T. (1979), "Foundation Investigation Report for Fairchild Creek Twin Bridges 15.0 km West of Hwy 2". Engineering Materials Office Report, Ministry of Transportation and Communications, 8 pp.

Kamphuis, J.W. and K.R. Hall, (1983), "Initiation of Erosion of Consolidated Cohesive Materials by Unidirectional Current", *Journal of the Hydraulics Division* , American Society of Civil Engineers, (ASCE), Vol. 109, No. 1, pp. 49-61.

Kamphuis, J.W. (1983), "Erodibility Tests on Till Samples from Lake Erie," Report prepared for the Deputy Attorney General of Canada (Alton et al. v. Her Majesty), 21 pp.

Kamphuis, J.W. (1983), "On the Erosion of Consolidated Clay Material by a Fluid Containing Sand", *Canadian Journal of Civil Engineering*, Vol. 10, No. 2, pp. 213-231.

Lefebvre, G., K Rohan, and S. Douville, (1985), "Erosivity of Natural Intact Structured Clay: Evaluation", *Canadian Geotechnical Journal*, Vol. 22, No. 4, pp. 508-517.

Lefebvre, G., K. Rohan, and J.P. Milette, (1986), "Erosivity of Natural Intact Clay: Influence of the Natural Structure", *Canadian Geotechnical Journal*, Vol. 23, No. 4, pp. 427-434.

Neil, C.R. (1972), *Guide to Bridge Hydraulics*, University of Toronto Press, 191 pp.

Table 1/ Competent Velocities for Cohesive Materials (m/s)
(from Neil (1972), p. 92)

Soil Scourability	Low	Medium	High
Depth of Flow (m)		Competent Velocity (m/s)	
1.5	0.6	1.0	1.8
3	0.65	1.2	2.0
6	0.7	1.3	2.3
15	0.8	1.5	2.6

Table 2/ Critical Shear Stresses
(Neil (1972), p. 185)

Scourability	High	Medium	Low
Shear Stress (psf)	0.035	0.12	0.35
Shear Stress (Pa)	1.67	5.74	16.75

Table 3.1/ Erosion-Shear Stress Relationship

Test	Location	Sample Number	Face Tested	Notes	Elapsed Time (hrs)	Fluid Velocity (m/s)	Shear Velocity (m/s)	Shear Stress (Pa)	Eroded Depth (mm)
H1T2	Humber River	1	TOP		2	0.96	0.051	2.64	0.42
H1T4					4	1.32	0.063	4.00	6.22
H1B2	Humber River	1	BOTTOM		2	0.50	0.031	0.93	0.09
H1B4					4	0.90	0.045	2.01	4.09
H2B2	Humber River	2	BOTTOM		2	1.42	0.070	4.87	0.19
H2B4					4	2.10	0.097	9.34	1.72
H2B6.5					6.5	2.91	0.129	16.51	8.98
H3T2	Humber River	3	TOP		2	1.70	0.079	6.17	7.59
H3T4					4	1.85	0.007	7.56	9.76
H3B2	Humber River	3	BOTTOM		2	1.32	0.066	4.35	1.11
H3B6					6	1.78	0.004	7.03	8.64
H3B8					8	2.10	0.095	9.05	9.25
H3B10					10	2.22	0.100	10.01	9.67
H3B12					12	2.70	0.121	14.57	11.12
H5B1	Humber River	5	BOTTOM		1	0.58	0.032	1.02	0.30
H5B2					2	0.72	0.040	1.61	1.12
H5B6					6	1.10	0.053	2.75	2.52
H5B8					8	1.42	0.070	4.92	4.69
H5B10					10	1.98	0.094	8.82	8.16
H1T6S	Humber River	1	TOP	sand	6	0.76	0.038	1.46	4.77
H1T8S				sand	8	0.97	0.049	2.30	8.75
H2T2S	Humber River	2	TOP	sand	2	0.75	0.038	1.43	5.89
H2T4S				sand	4	0.92	0.050	2.46	14.15
H5T2S	Humber River	5	TOP	sand	2	0.09	0.050	2.46	5.00
H5T4S				sand	4	1.10	0.057	3.28	12.61
F1B2	Fairchild Creek	1	BOTTOM		2	0.66	0.037	1.37	0.26
F1B4					4	0.97	0.051	2.55	1.35
F1B6					6	1.36	0.071	5.05	6.79
F3T2	Fairchild Creek	3	TOP		2	0.95	0.049	2.43	0.05
F3T4					4	1.61	0.076	5.72	1.03
F3T5					5	2.01	0.095	9.01	5.09
F4B2	Fairchild Creek	4	BOTTOM		2	1.63	0.078	6.01	0.95
F4B6					6	2.06	0.094	8.86	7.57
F4B8					8	2.15	0.098	9.61	11.15
F4B10					10	2.29	0.103	10.57	13.90
F1T2S	Fairchild Creek	1	TOP	sand	2	0.92	0.048	2.30	0.17
F1T3S				sand	3	1.19	0.059	3.49	16.24
F3B2S	Fairchild Creek	3	BOTTOM	sand	2	0.61	0.038	1.42	3.22
F3B6S				sand	6	0.90	0.048	2.33	13.39
F4T2S	Fairchild Creek	4	TOP	sand	2	0.71	0.041	1.71	5.68
F4T4S				sand	4	0.88	0.043	1.86	8.61
F4T6S				sand	6	1.00	0.053	2.76	14.19

Table 3.2/ Erosion-Shear Stress Relationship

Test	Location	Sample Number	Face Tested	Notes	Elapsed Time (hrs)	Fluid Velocity (m/s)	Shear Velocity (m/s)	Shear Stress (Pa)	Eroded Depth (mm)
D1B2	Dingman Creek	1	BOTTOM		2	0.62	0.036	1.27	3.10
D1B4					4	0.99	0.049	2.43	5.76
D1B6					6	1.20	0.060	3.58	6.86
D1B7					7	1.62	0.077	5.91	12.53
D2T2	Dingman Creek	2	TOP		2	0.60	0.034	1.17	0.77
D2T4					4	0.80	0.044	1.90	1.26
D2T6					6	0.98	0.052	2.73	2.46
D2T8					8	1.25	0.063	3.95	4.32
D2T9					9	1.49	0.073	5.37	7.31
D1T2S	Dingman Creek	1	TOP	sand	2	0.91	0.049	2.35	3.83
D1T4S				sand	4	1.05	0.057	3.23	11.11
D1T6S				sand	6	1.25	0.060	3.63	15.82
C1B2	Collins Creek	1	BOTTOM		2	0.90	0.046	2.11	2.24
C1B4					4	1.05	0.052	2.67	6.89
C2B1	Collins Creek	2	BOTTOM		1	0.55	0.029	0.84	0.47
C2B3					3	0.85	0.045	1.98	1.44
C2B5					5	1.00	0.051	2.57	2.57
C2T1.5	Collins Creek	2	TOP		1.5	1.03	0.050	2.48	3.09
C3T2	Collins Creek	3	TOP		2	0.65	0.039	1.51	1.30
C3T4					4	0.90	0.047	2.22	1.49
C3T6					6	1.10	0.055	2.98	2.34
C4B2	Collins Creek	4	BOTTOM		2	0.52	0.032	1.00	1.13
C4B4					4	0.81	0.044	1.93	3.04
C4B6					6	0.96	0.051	2.61	5.23
C4B8					8	1.61	0.074	5.47	6.67
C4B10					10	2.35	0.104	10.80	8.30
C4T2S	Collins Creek	4	TOP	sand	2	0.60	0.033	1.07	0.77
C4T4S				sand	4	0.96	0.047	2.21	1.48

Table 4/ Summary of Soils Properties and Erodibility

Test	Gravel %	Sand %	Silt %	Clay %	OMC %	FMC %	LL %	PL %	PI %	PreCon Stress kPa	q kPa	Shear Strength kPa	Tau c Clear Pa	Tau c Sand Pa	a Clear cm/Pa	a Sand cm/Pa
C2B	1	19	57	23	29	43	41	22	19	190			0.60		1.30	
C3T	1	19	57	23	29					400			0.60		0.90	
C4B	1	19	57	23		32							0.60		2.80	
C4TS	11	19	57	23		32								0.60		
C8											320	160				
H1T	3	12	49	36	28	50	40	19	21	140	176	88	2.60		1.30	
H1B	3	12	49	36	28	50	40	19	21	140	176	88	0.70		3.00	
H2B	0	0	60	40	26	30	38	22	16	100	165	82	6.00		0.75	
H3T	0	15	42	43	28	31	42	19	23	150	108	54	1.00		1.40	
H3B	0	15	42	43	28	31	42	19	23	150	108	54	4.00		2.50	
H3B	0	2	61	37	27	27	38	20	18			0	1.00		1.25	
H2TS	0	0	60	40	26	30	38	22	16	100	165	82		0.70		8.00
H3TS	0	2	61	37	27	27	38	22	16			0		1.90		9.50
F1B	0	8	56	36	27	31	27	22	5			0	1.30		1.75	
F2					24						200	100				
F3T	0	2	72	26	27	37	32	19	13	140	139	70	5.00		1.10	
F4B	0	5	76	19	26	28	36	16	20	236	277	139	6.00		2.90	
F1TS	0	8	56	36	27	31	27	22	5			0		1.10		6.80
F3BS	0	2	72	26	27	37	32	19	13	140	139	70		1.10		9.50
F4TS	0	5	76	19	26	28	36	16	20	236	237	119		0.80		7.20
D1B	5	13	57	25	14	21	30	17	13	260	424	212	0.00		2.00	
D2T	0	5	70	25	19	21	27	16	11	300	228	114	1.20		1.80	
D3	4	18	56	22	19		26	14	12	230	116	58				
D4	0	14	63	23	18		30	17	13							
D8					18						326	163				
D1TS	5	13	57	25	14	21	30	17	13	260	424	212		2.00		9.50

LEGEND

C	Colline Creek sample
H	Humber River sample
F	Fairchild Creek sample
D	Dingman Creek Sample
C4TS	Colline Creek - 4 hrs - Top of sample - Sand Suspension
OMC	Original Water Content
FMC	Final Water Content
LL	Liquid Limit
PL	Plastic Limit
PI	Plasticity Index
q	Unconfined Compressive Strength
TAU c	Critical value of shear stress at which soil begins to erode
a	Slope of the relationship between erosion and shear stress

Table 5/ Minerolgical Composition

Soil	Smectite	Chlorite	Muscovite-illite Biotite-illite	Amphibole	Quartz	Feldspar
Collins Creek	5	4	20	15	30	27
Humber River	-	10	29	-	50	11
Fairchild Creek	5	10	31	7	36	11
Dingman Creek	-	11	28	9	38	14

Figures 1 and 2/ High Speed Flume-Tunnel and Soil Press

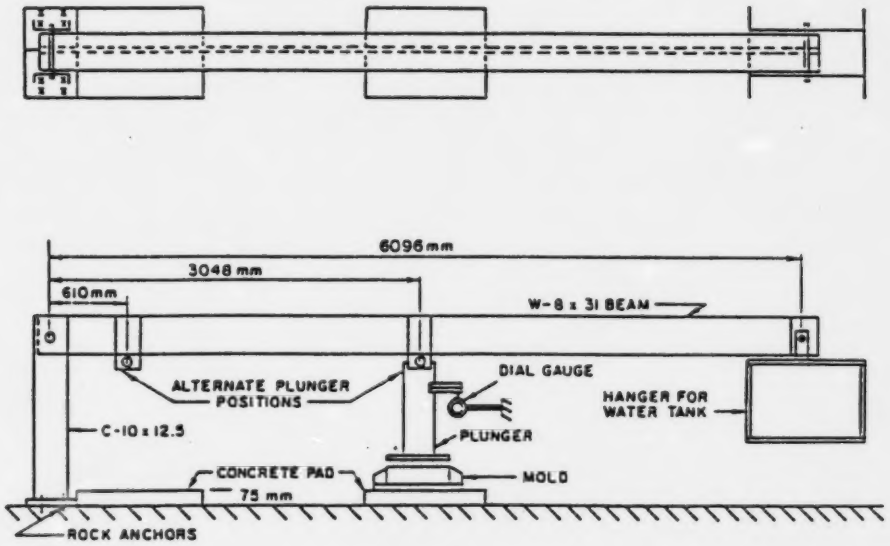


FIGURE 1 SOIL PRESS

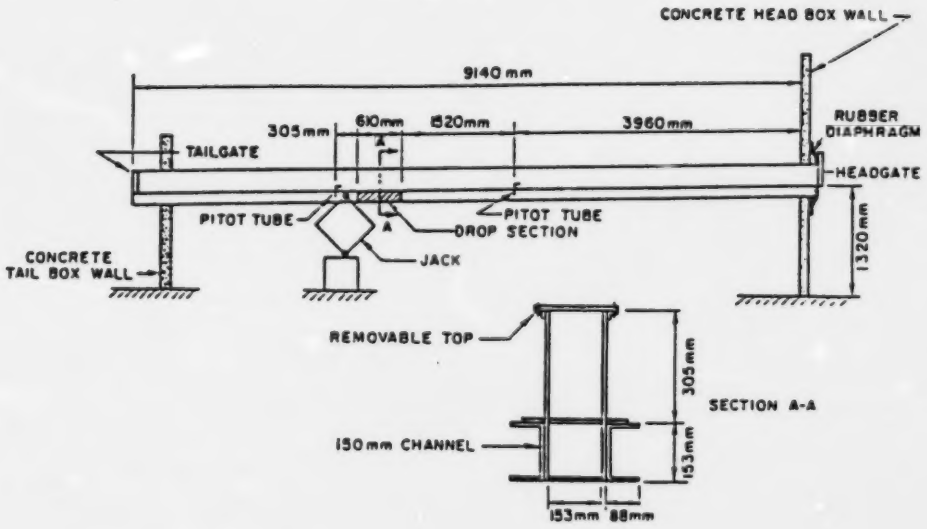


FIGURE 2 HIGH SPEED FLUME - TUNNEL

Figure 3/ Shields Diagram

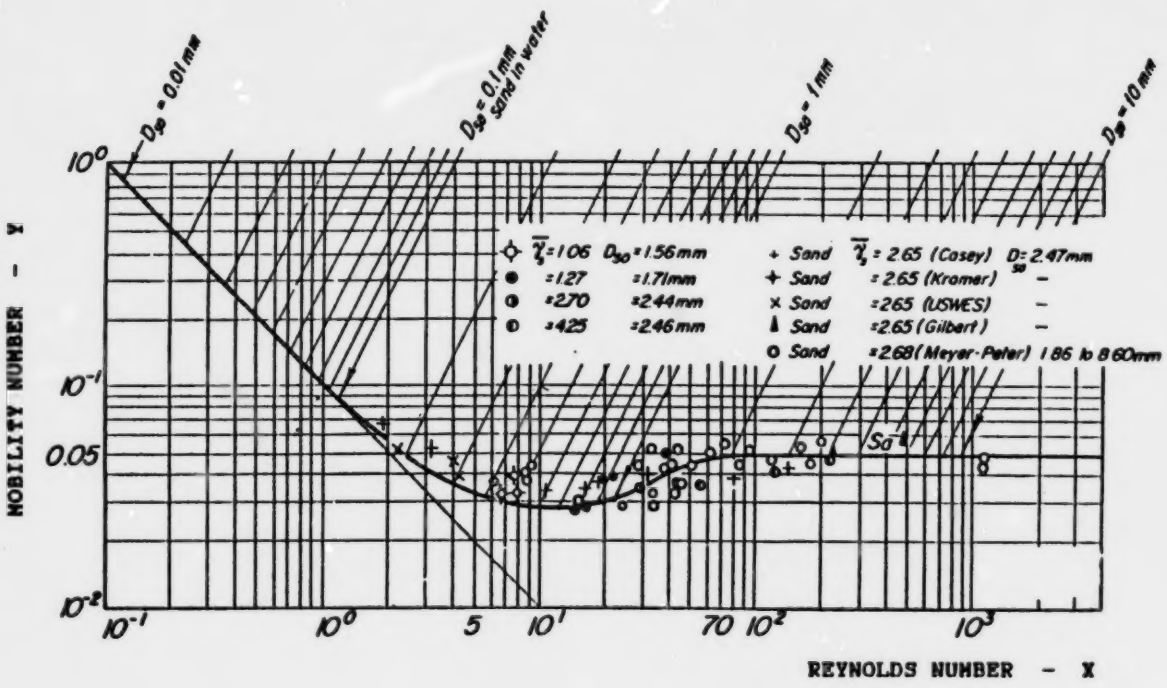


Figure 4/ Measuring Template

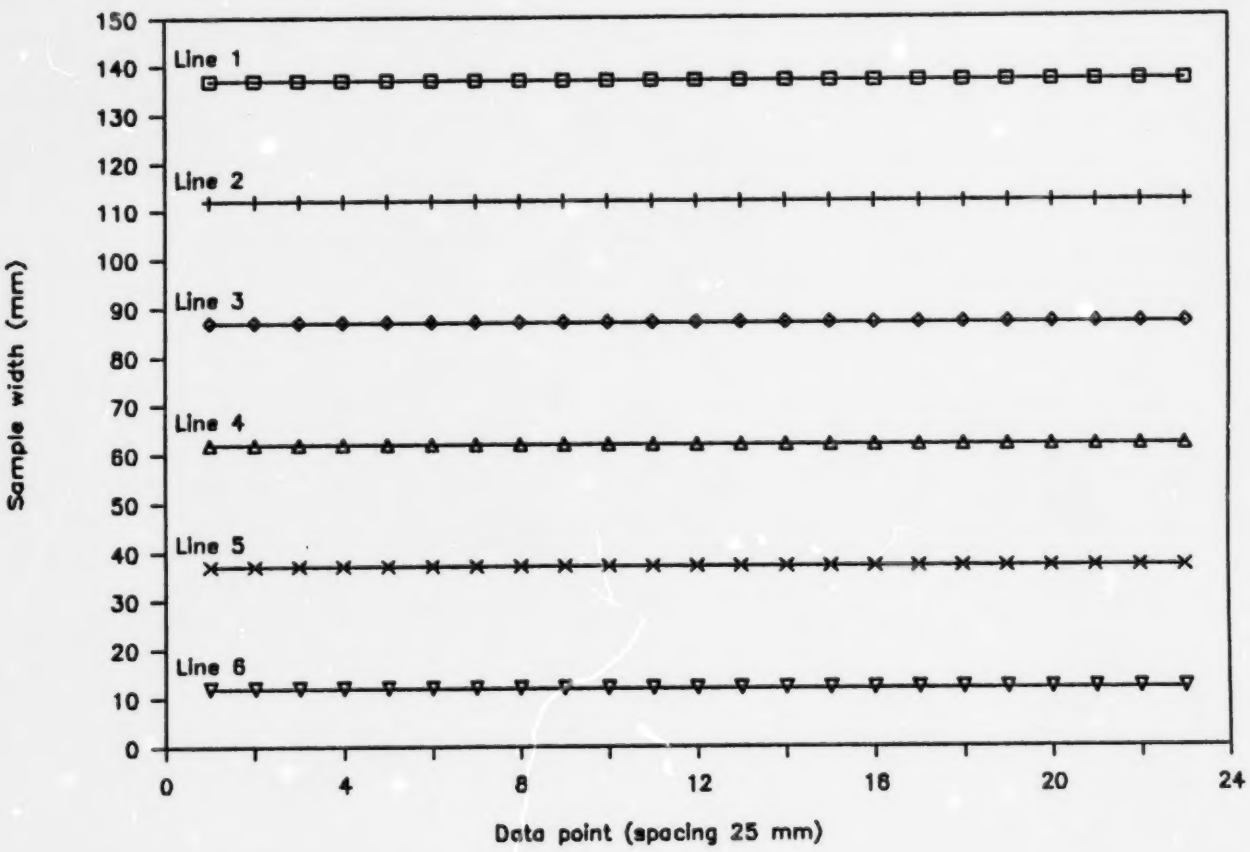


Figure 5/ Erosion Depth-Shear Stress

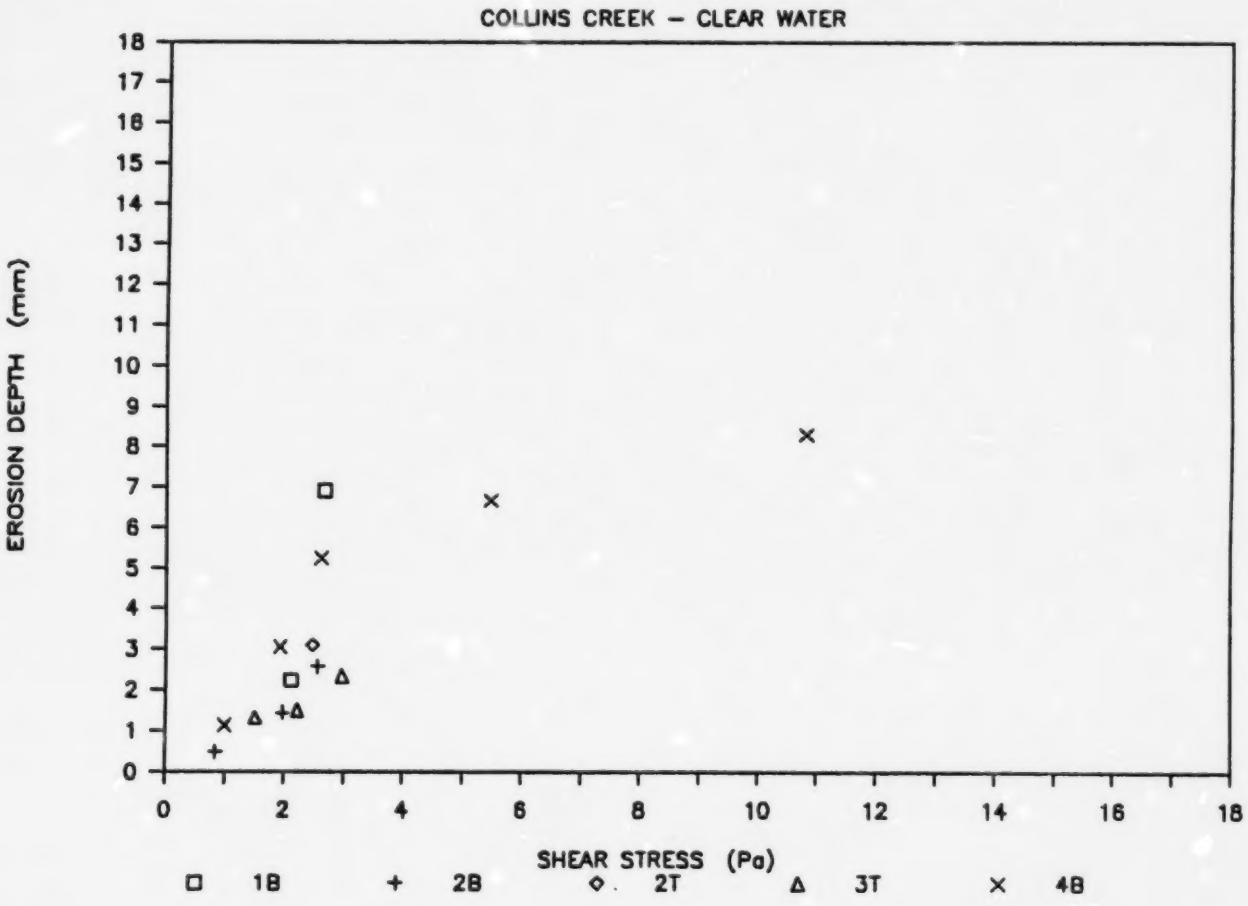


Figure 5a/ Erosion Depth-Shear Stress

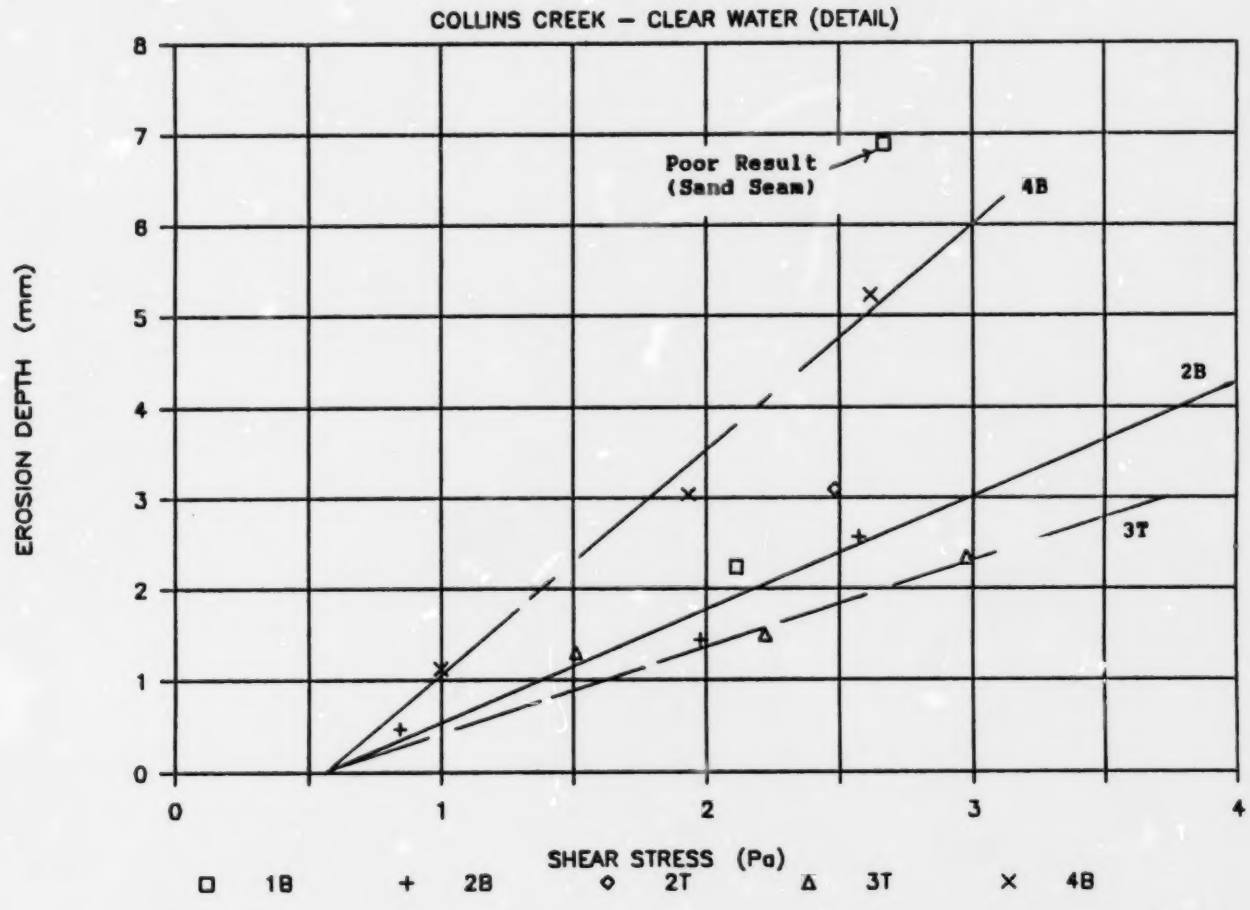


Figure 6/ Erosion Depth-Shear Stress

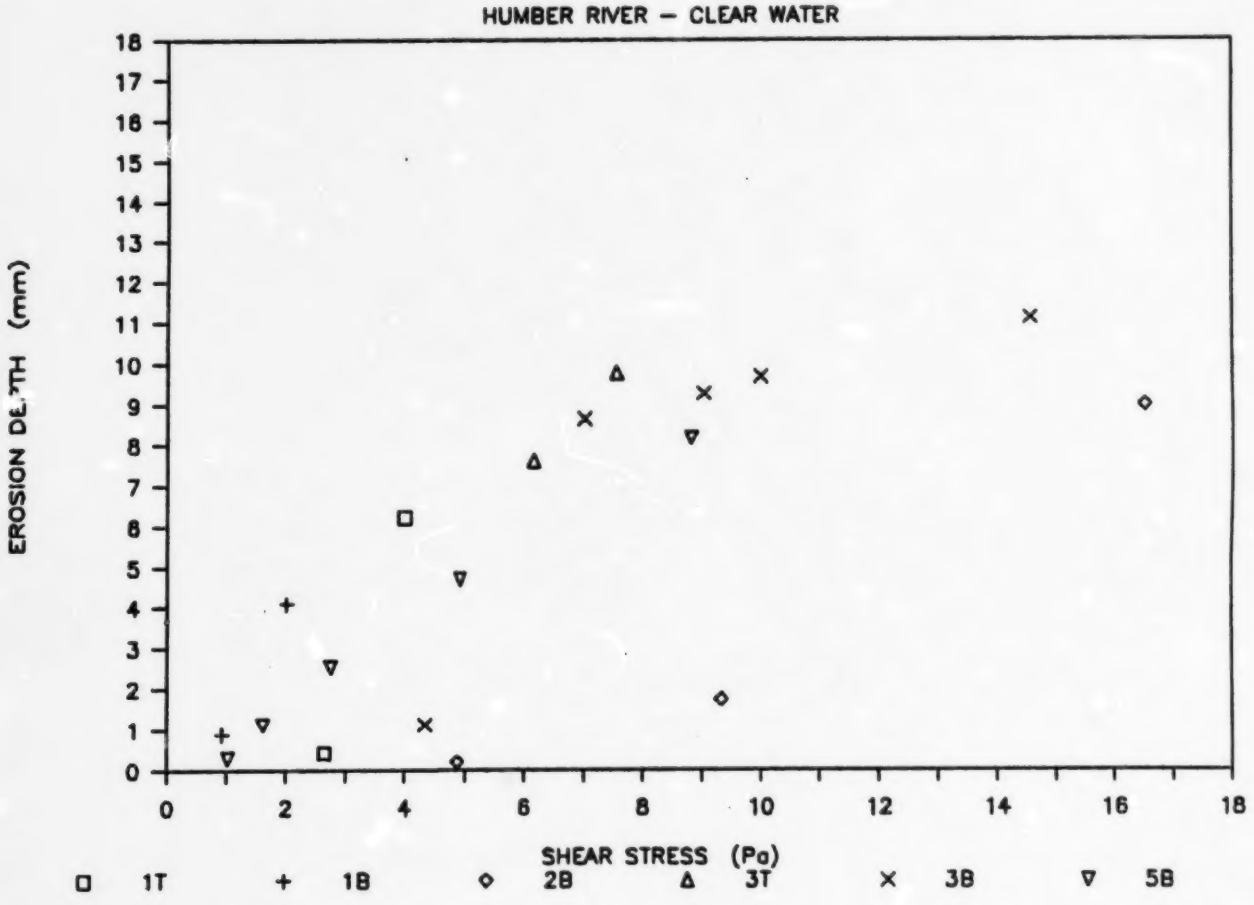


Figure 7/ Erosion Depth-Shear Stress

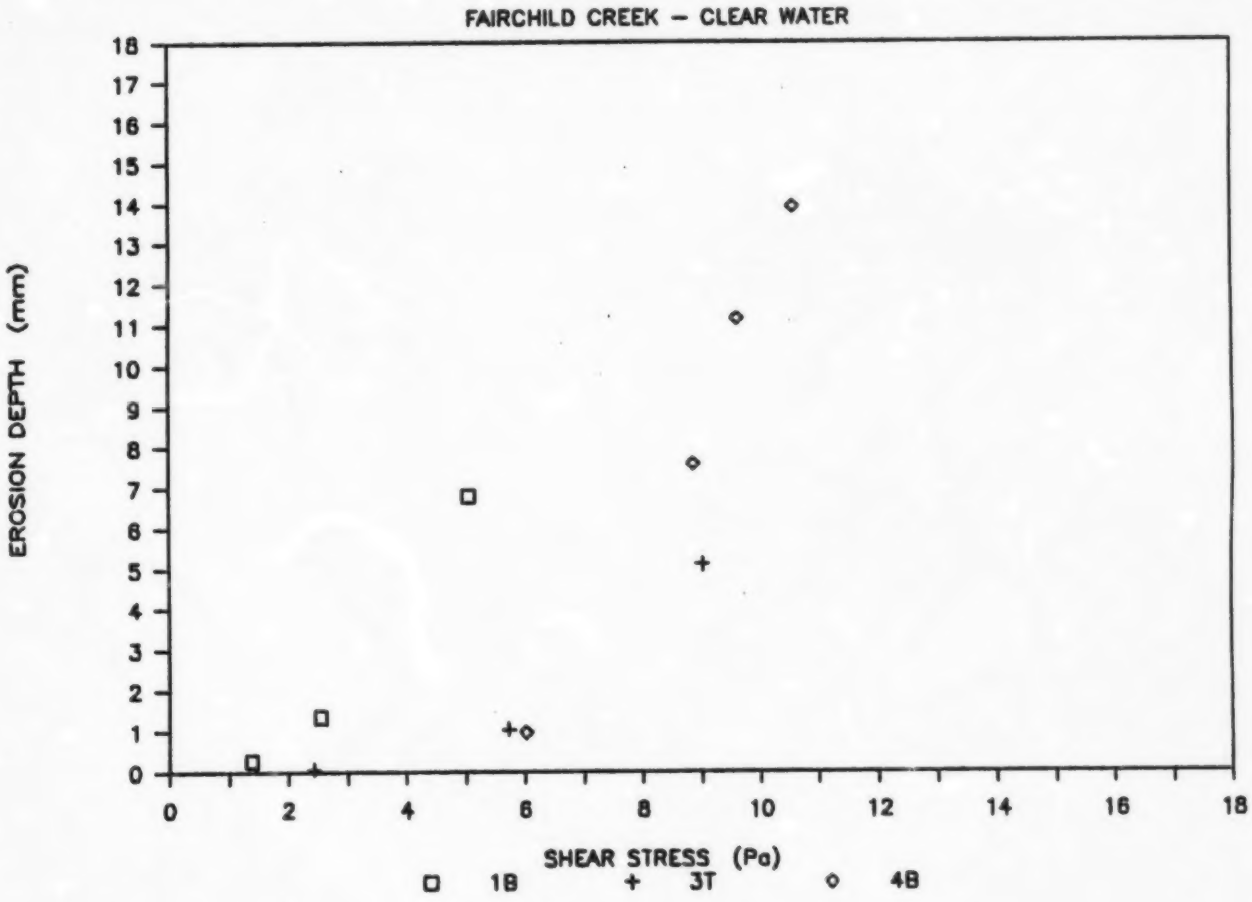


Figure 8/ Erosion Depth-Shear Stress

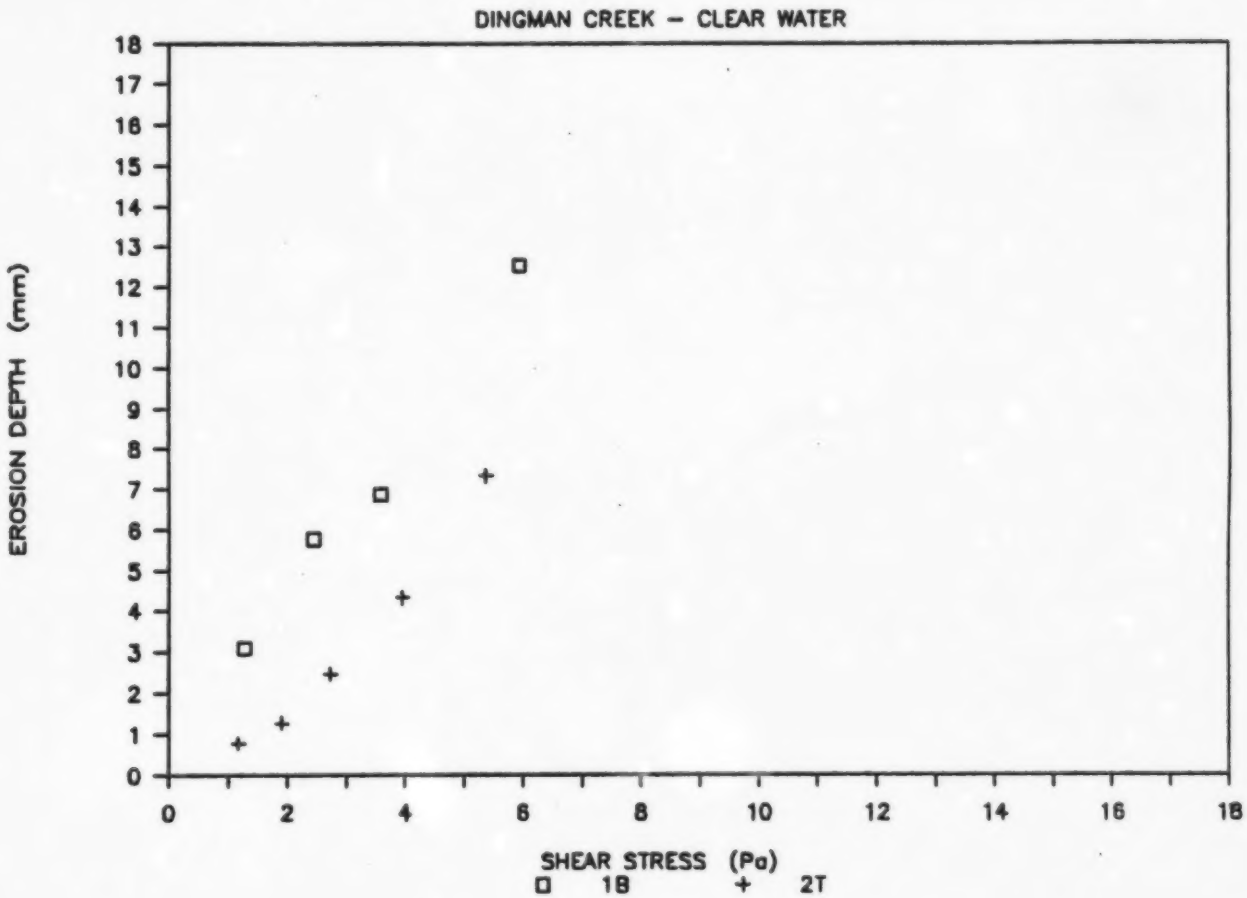


Figure 9/ Erosion Depth-Shear Stress

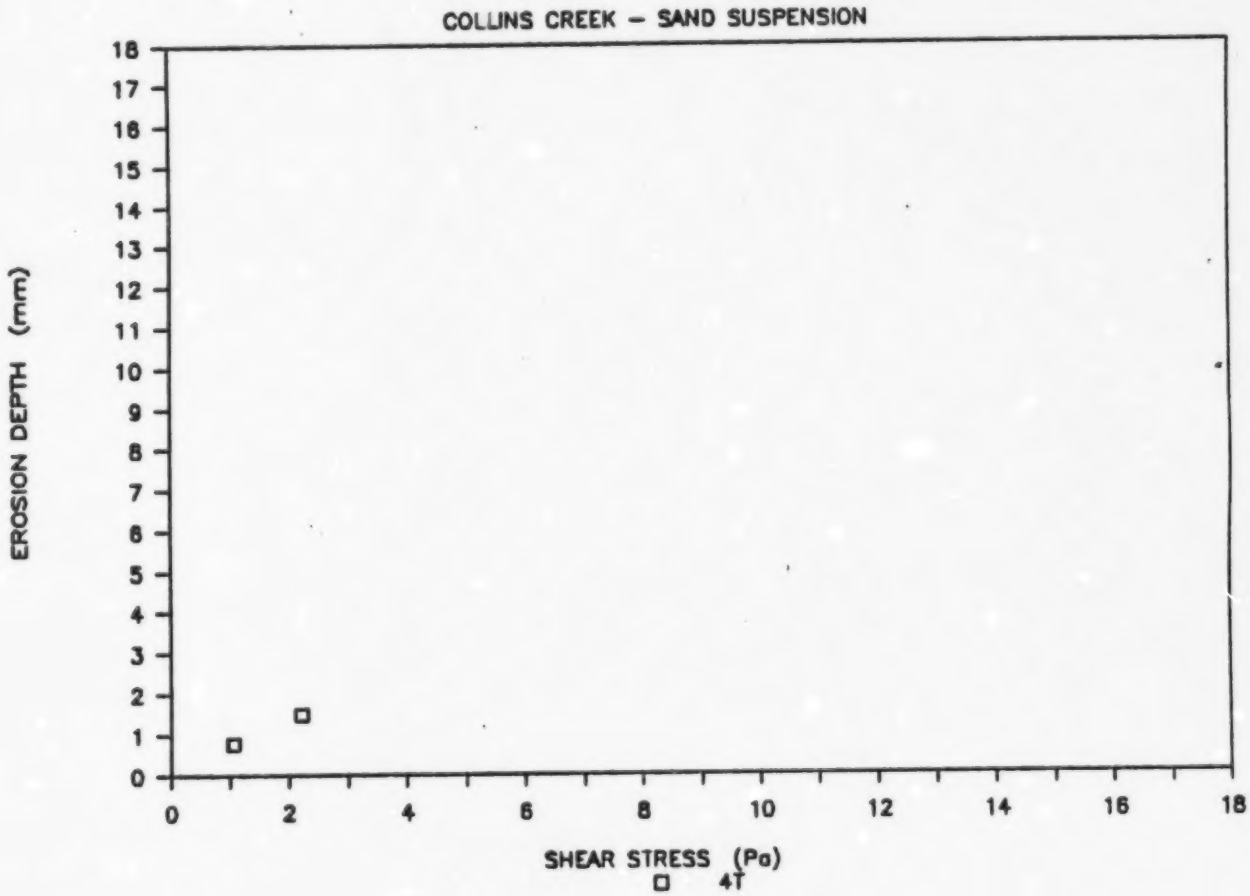


Figure 10/ Erosion Depth-Shear Stress

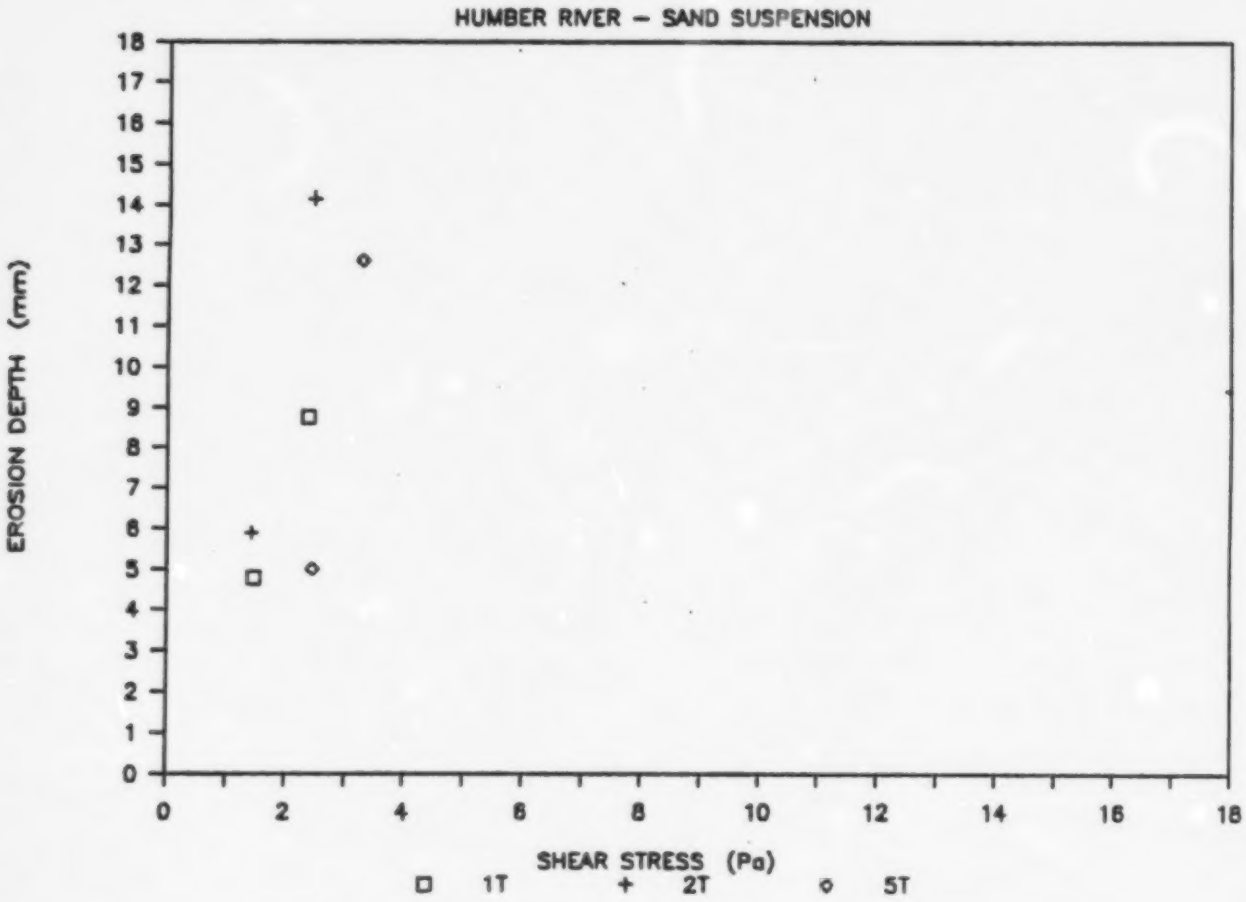


Figure 11/ Erosion Depth-Shear Stress

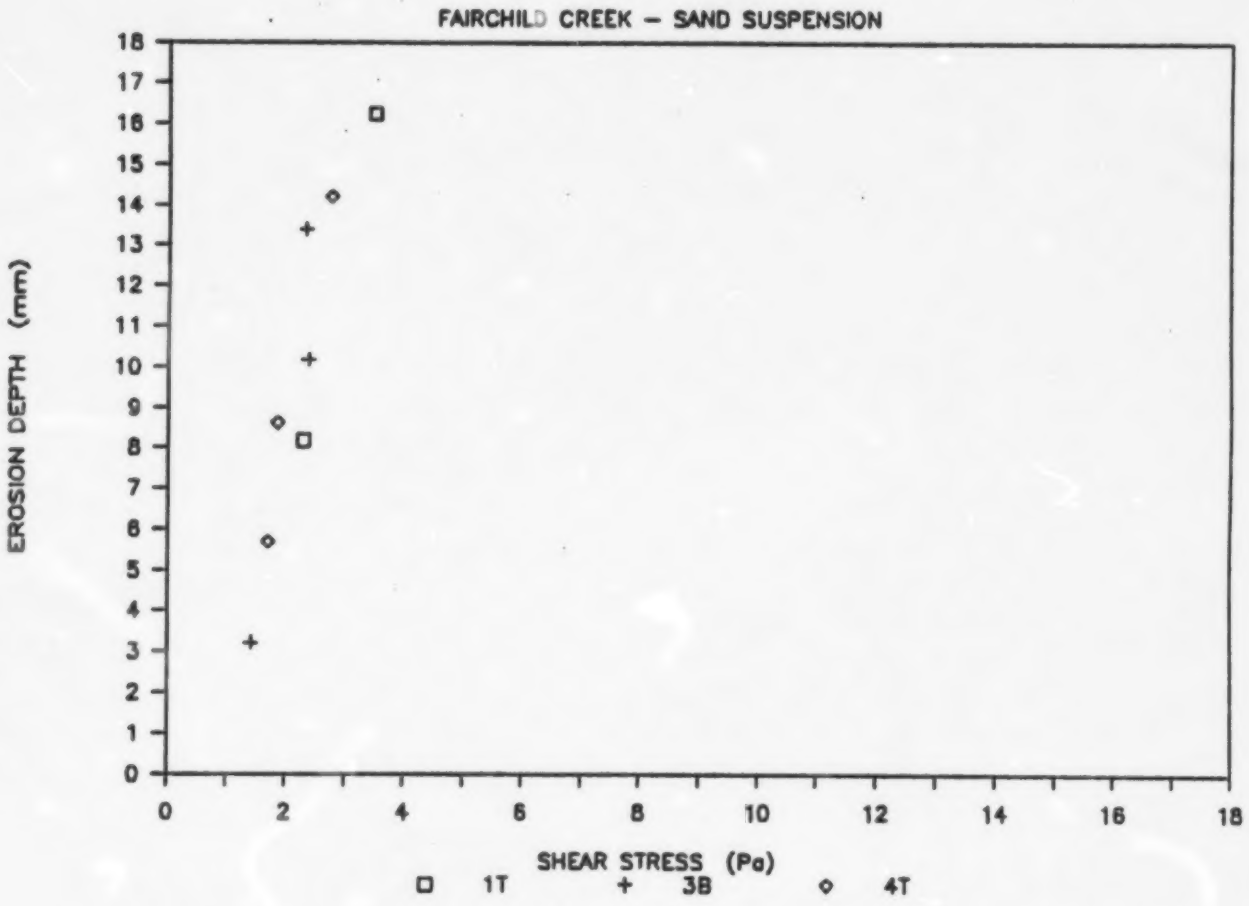


Figure 12/ Erosion Depth-Shear Stress

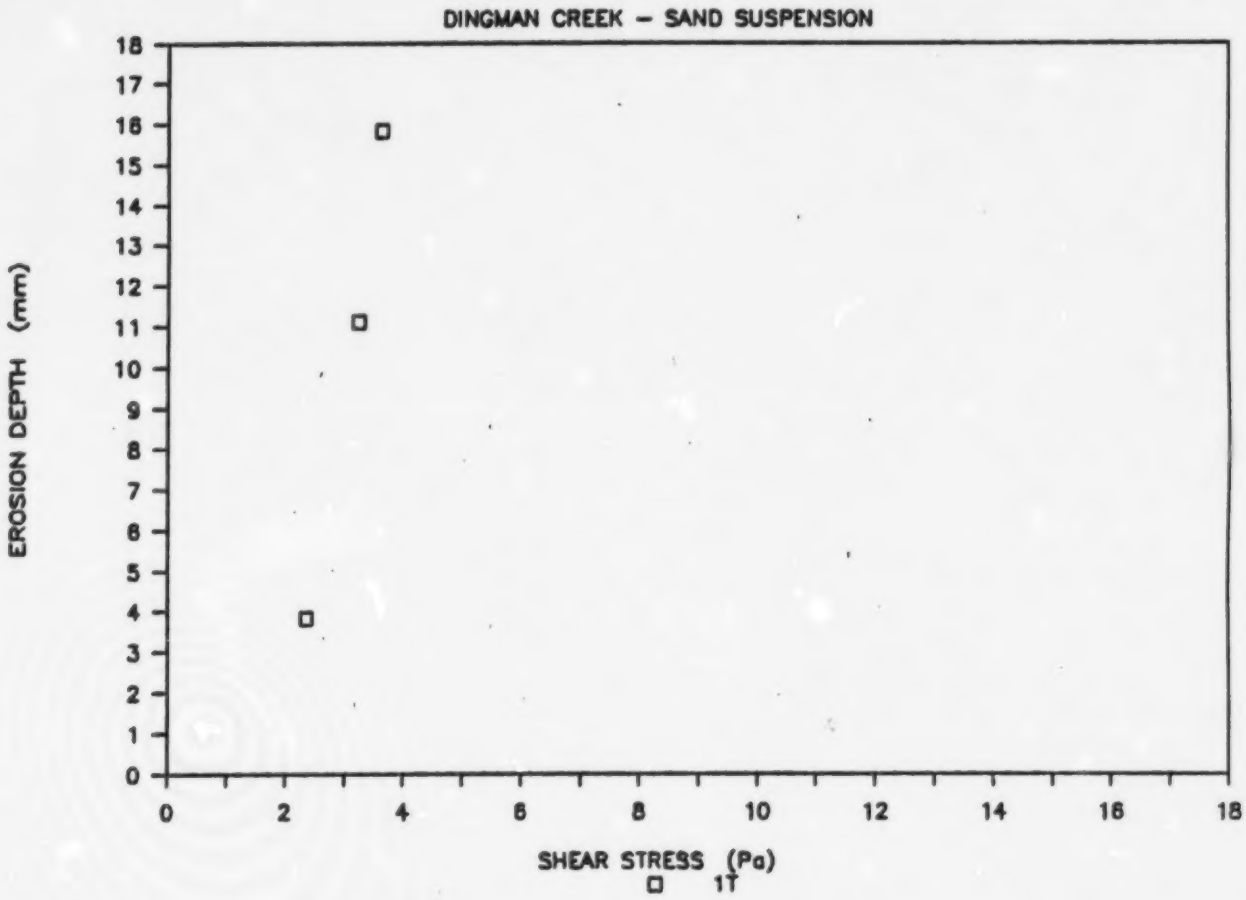


Figure 13/ Erosion Depth-Shear Stress

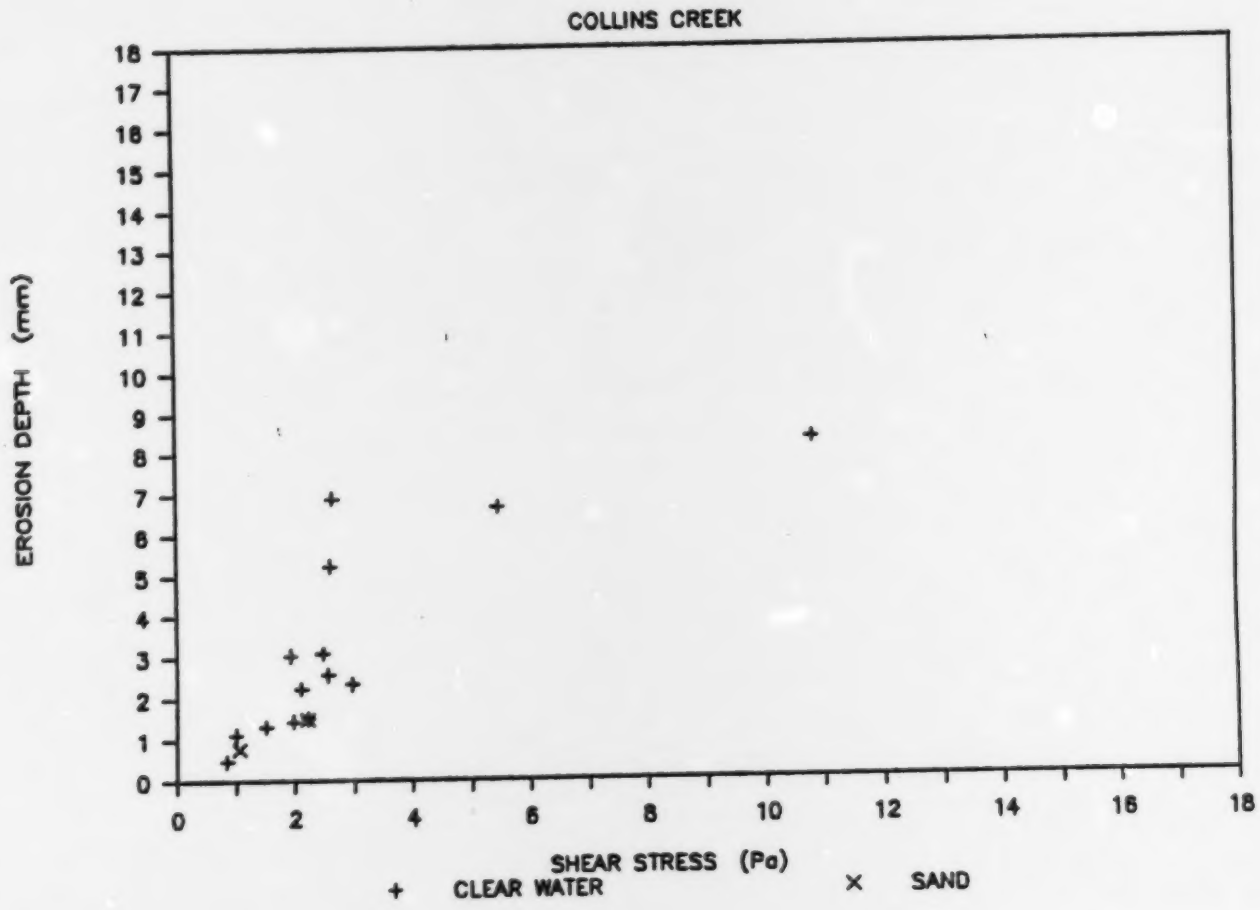


Figure 14/ Erosion Depth-Shear Stress

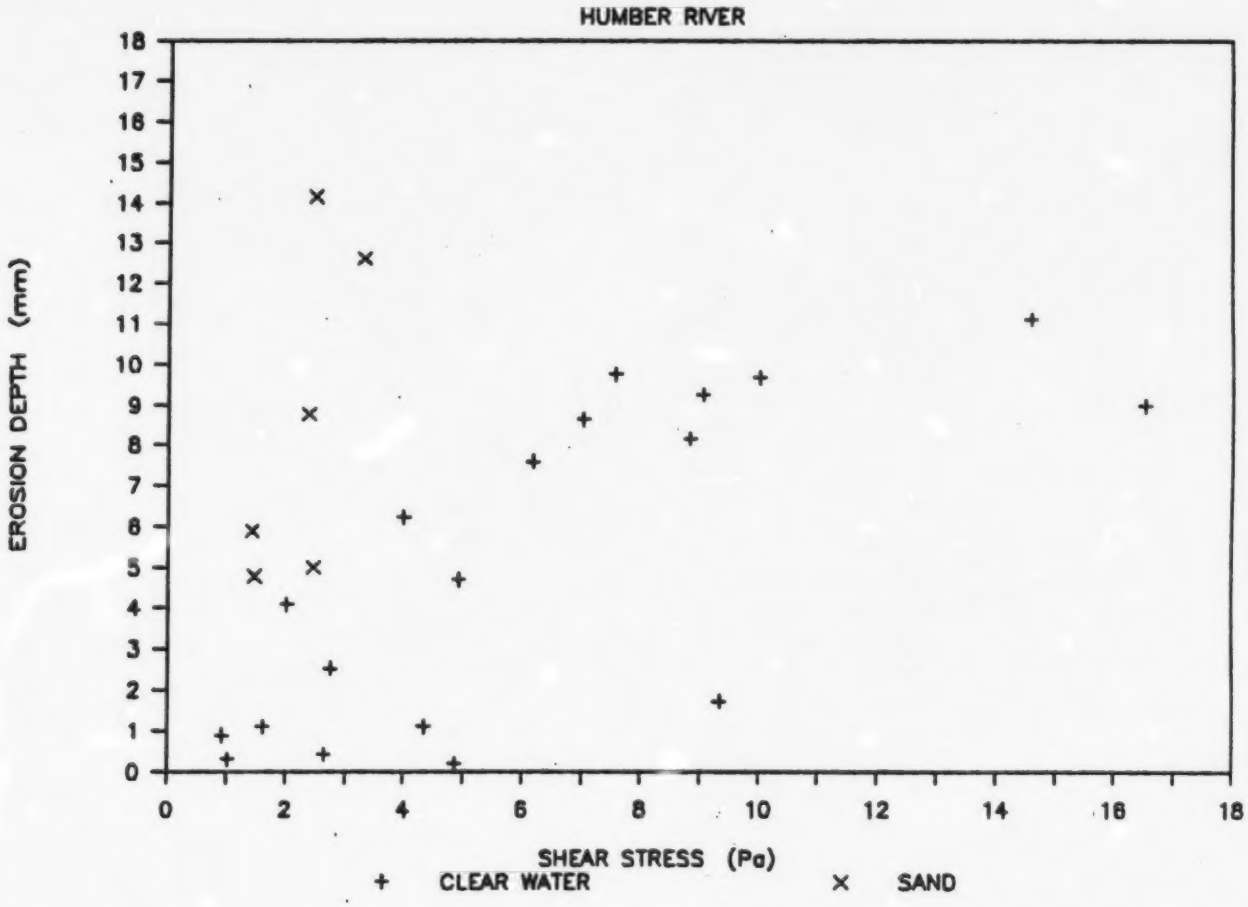


Figure 15/ Erosion Depth-Shear Stress

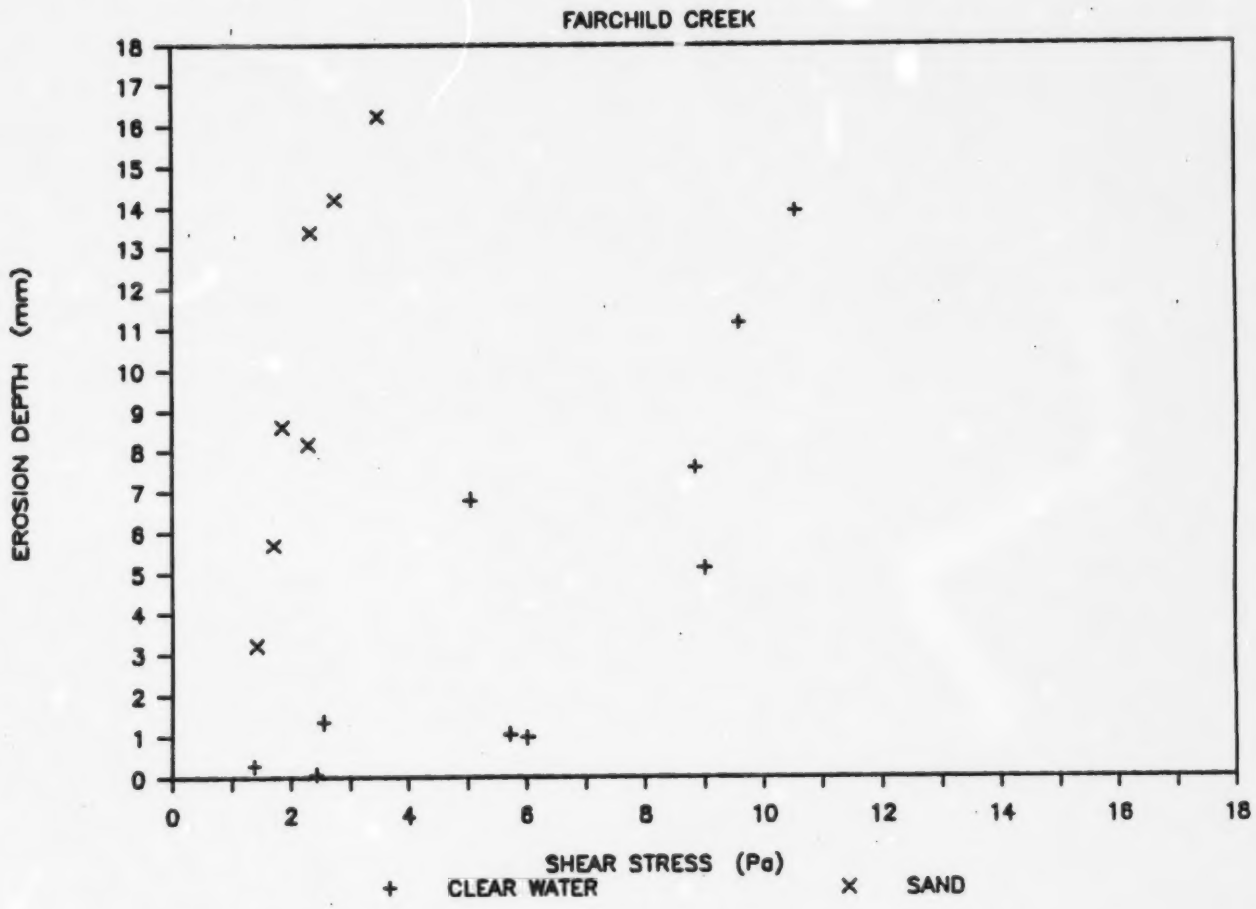


Figure 16/ Erosion Depth-Shear Stress

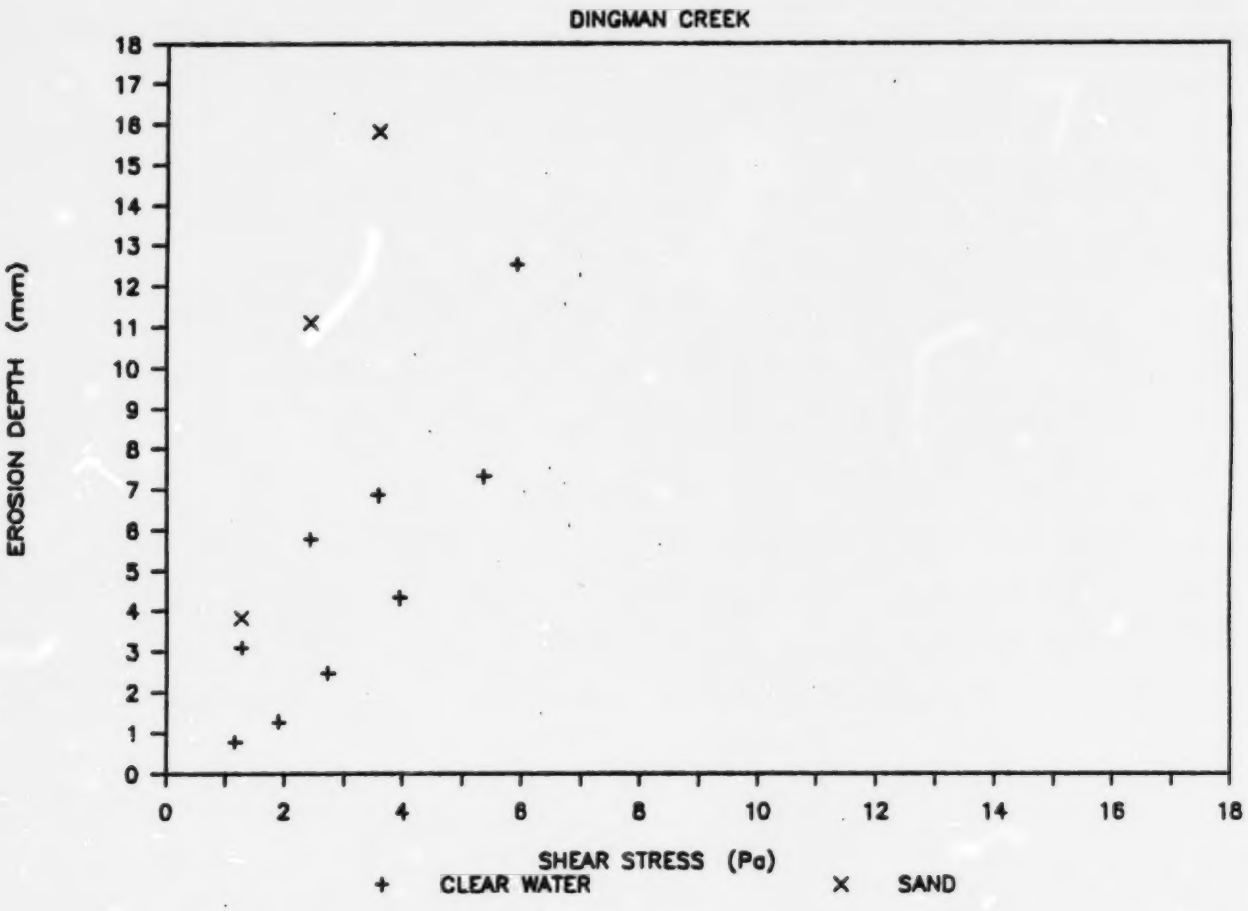


Figure 17/ Erosion Depth-Shear Stress

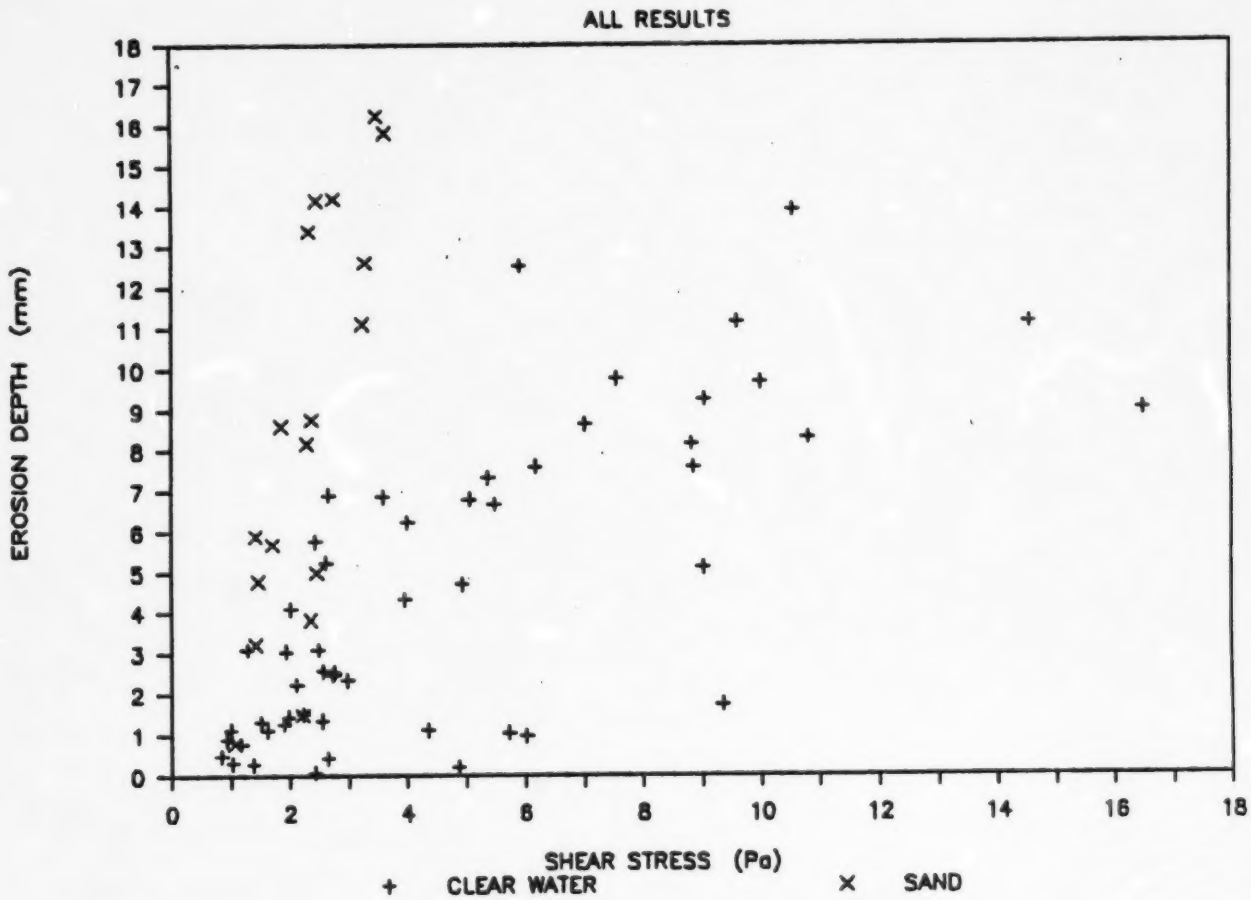


Figure 18/ Preconsolidation Stress vs Original Water Content

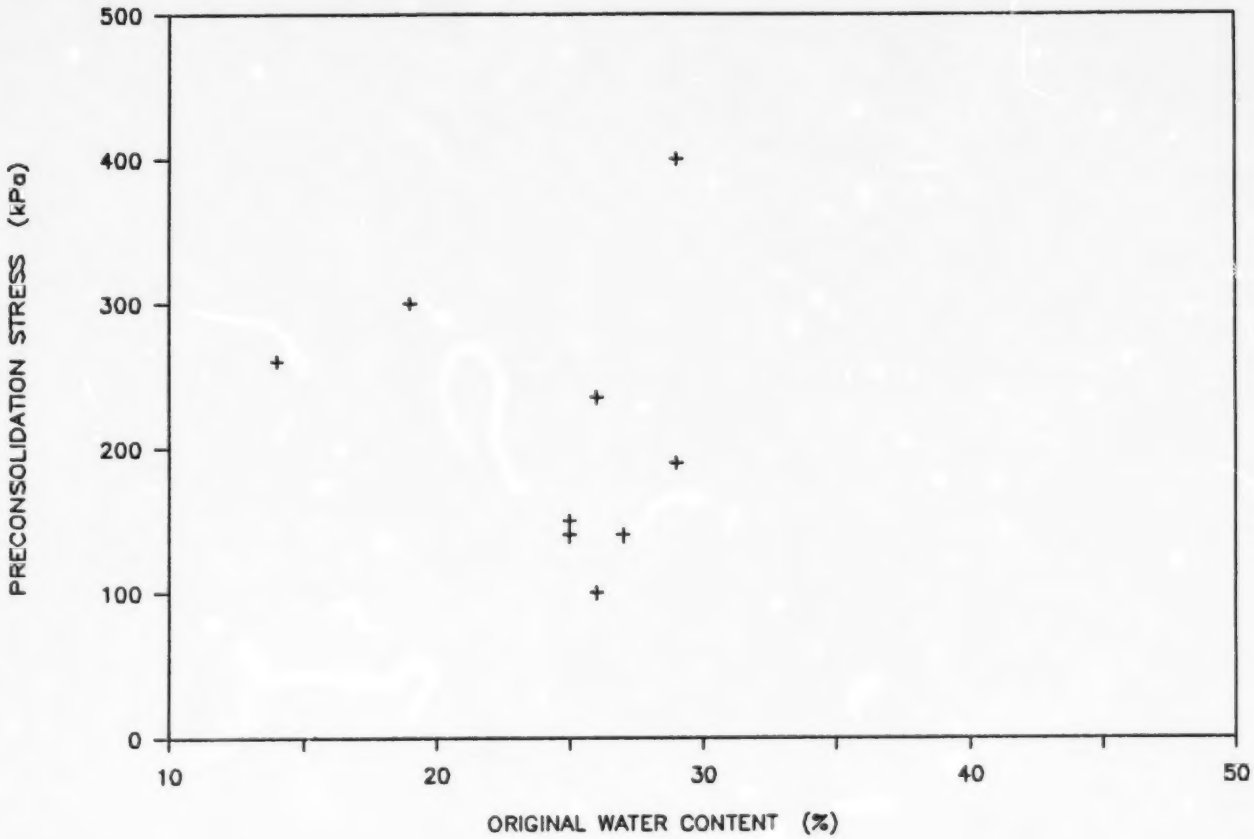


Figure 19/ Soil Sample

Figure 20/ Soil Sample

Figure 21/ Critical Shear Stress vs Original Water Content

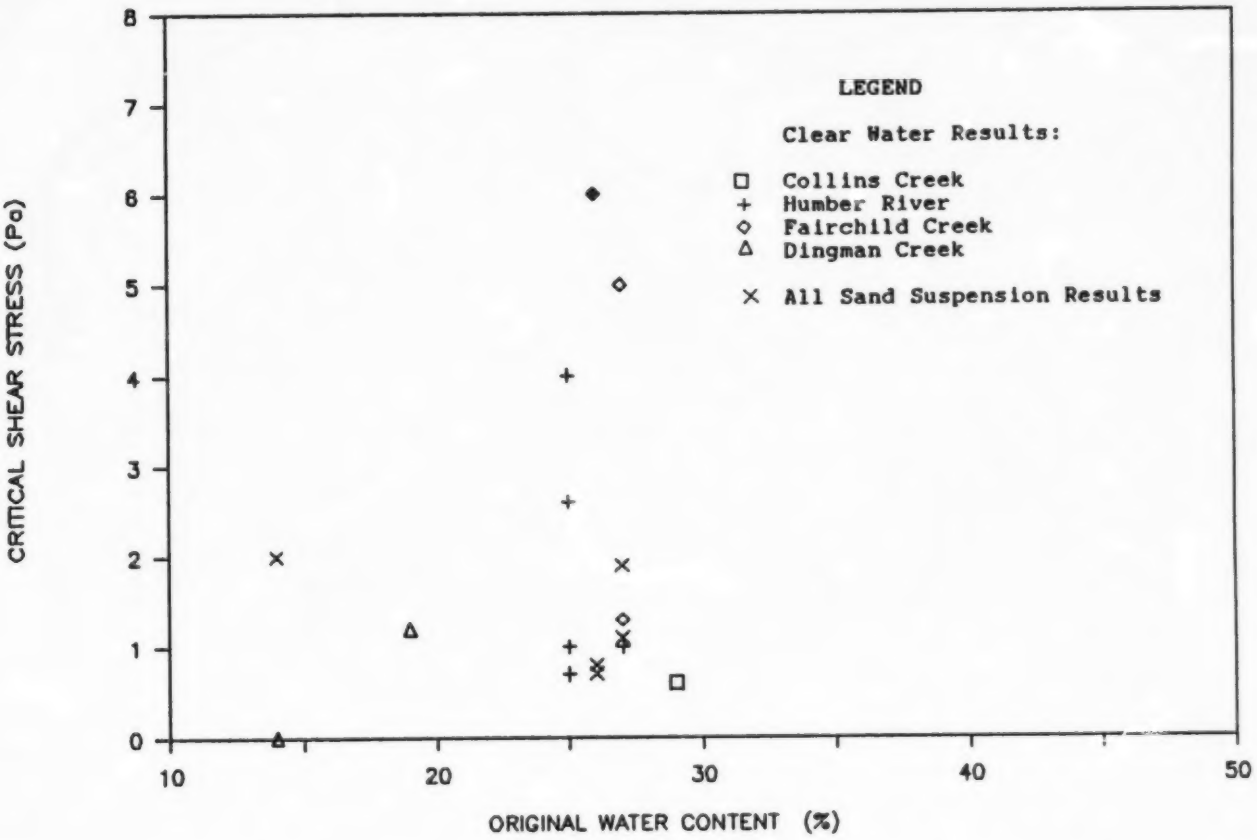


Figure 22/ Critical Shear Stress vs Final Water Content

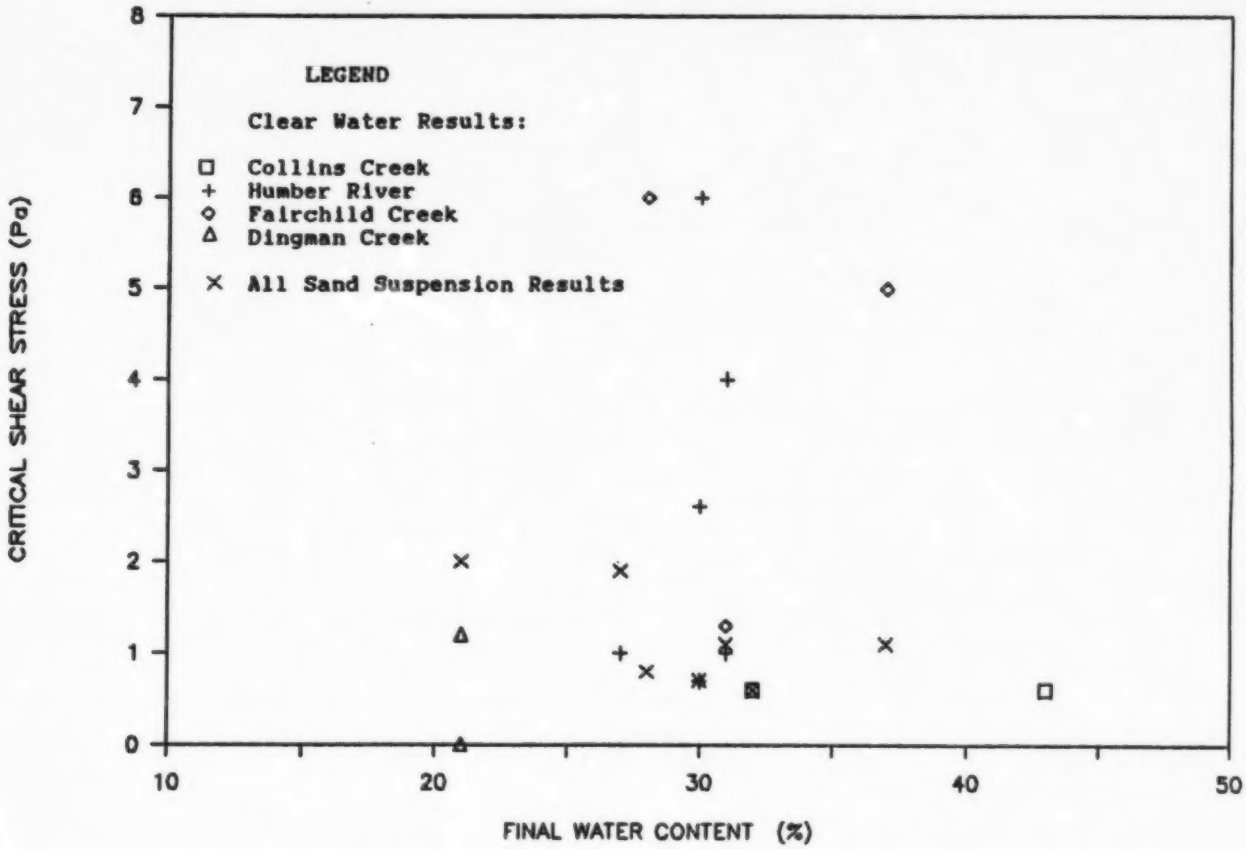


Figure 23/ Critical Shear Stress vs Clay Content

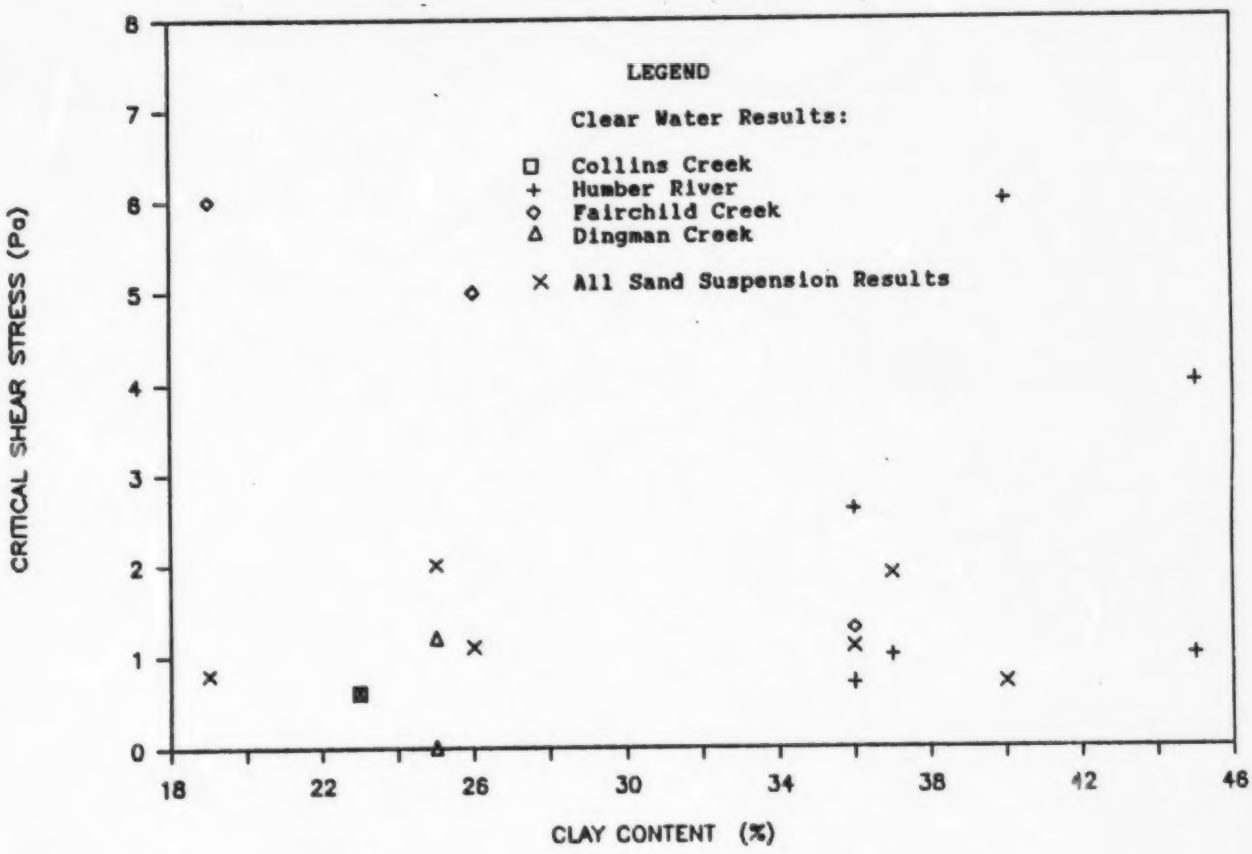


Figure 24/ Critical Shear Stress vs Plasticity Index

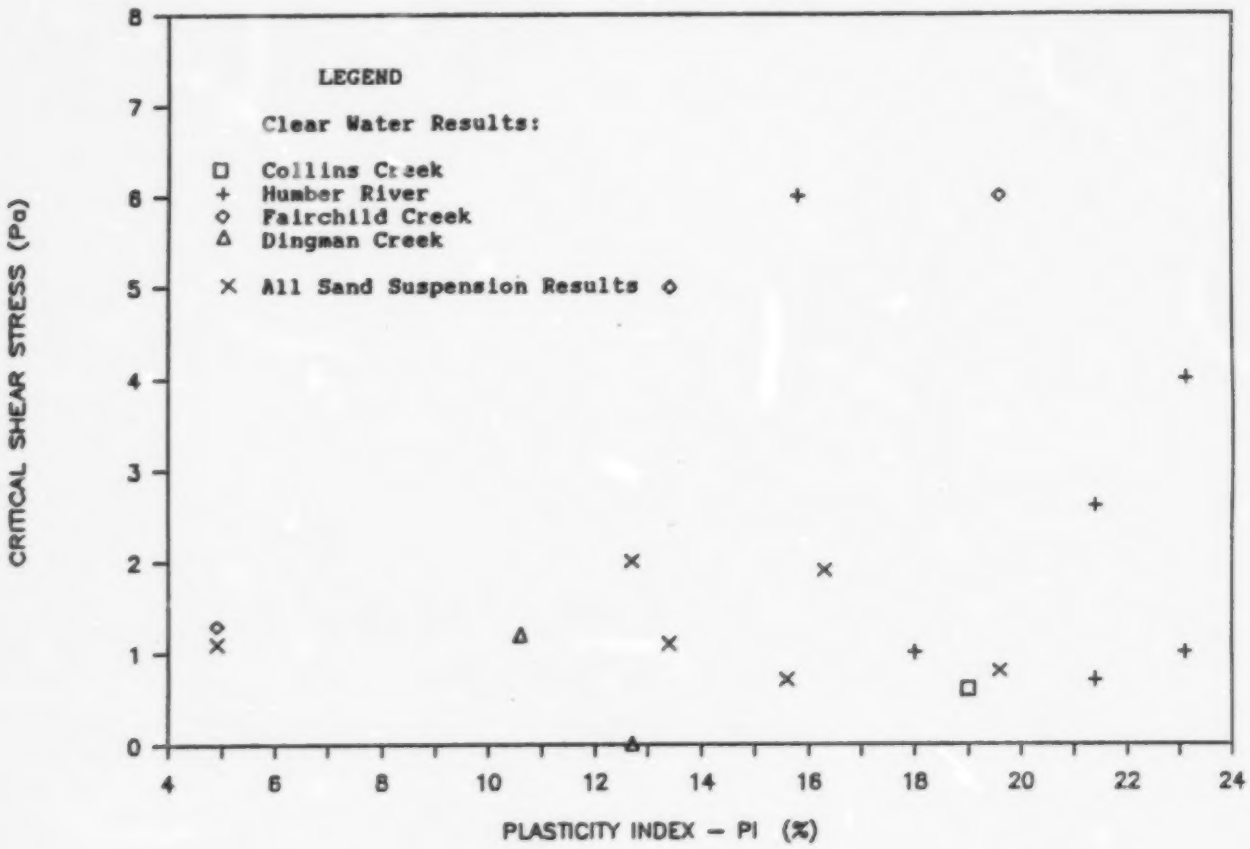


Figure 25/ Critical Shear Stress vs Precon Stress

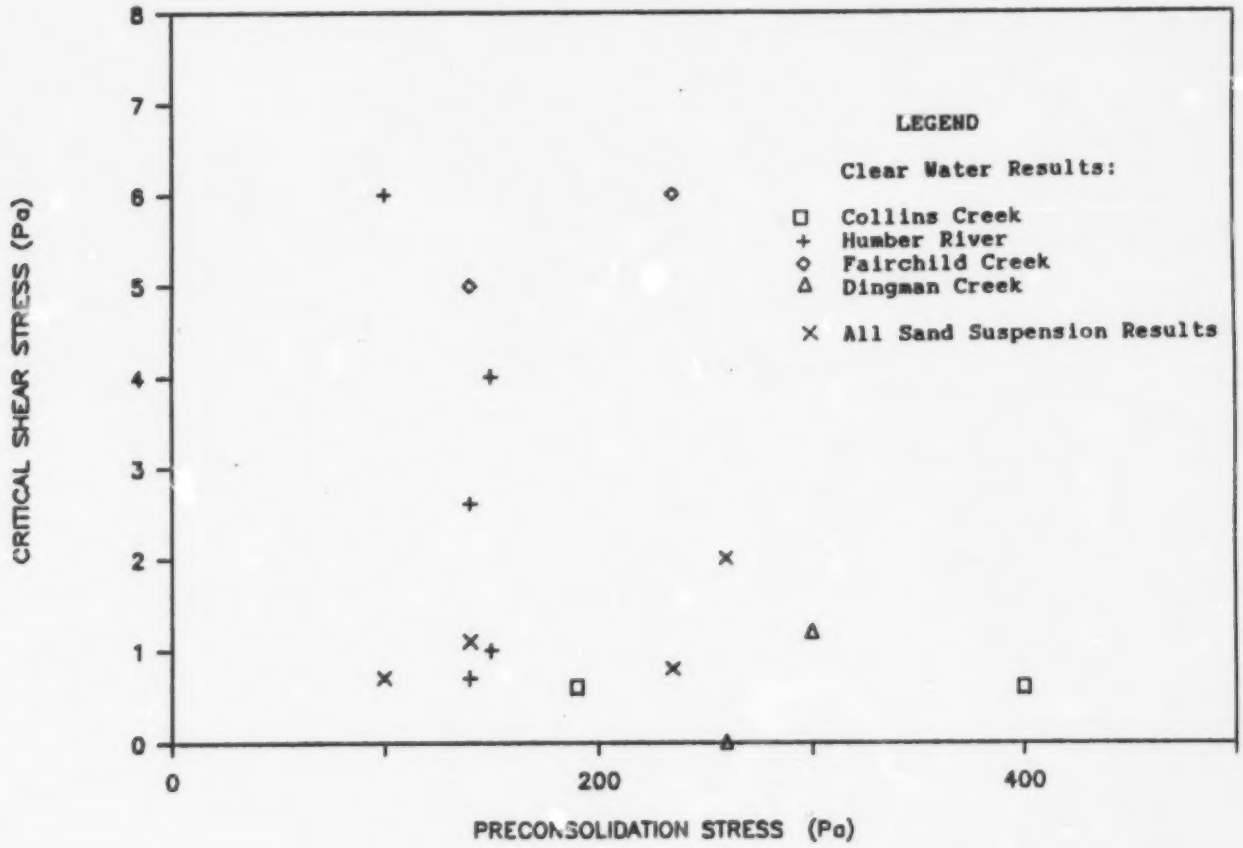


Figure 26/ Critical Shear Stress vs Shear Strength

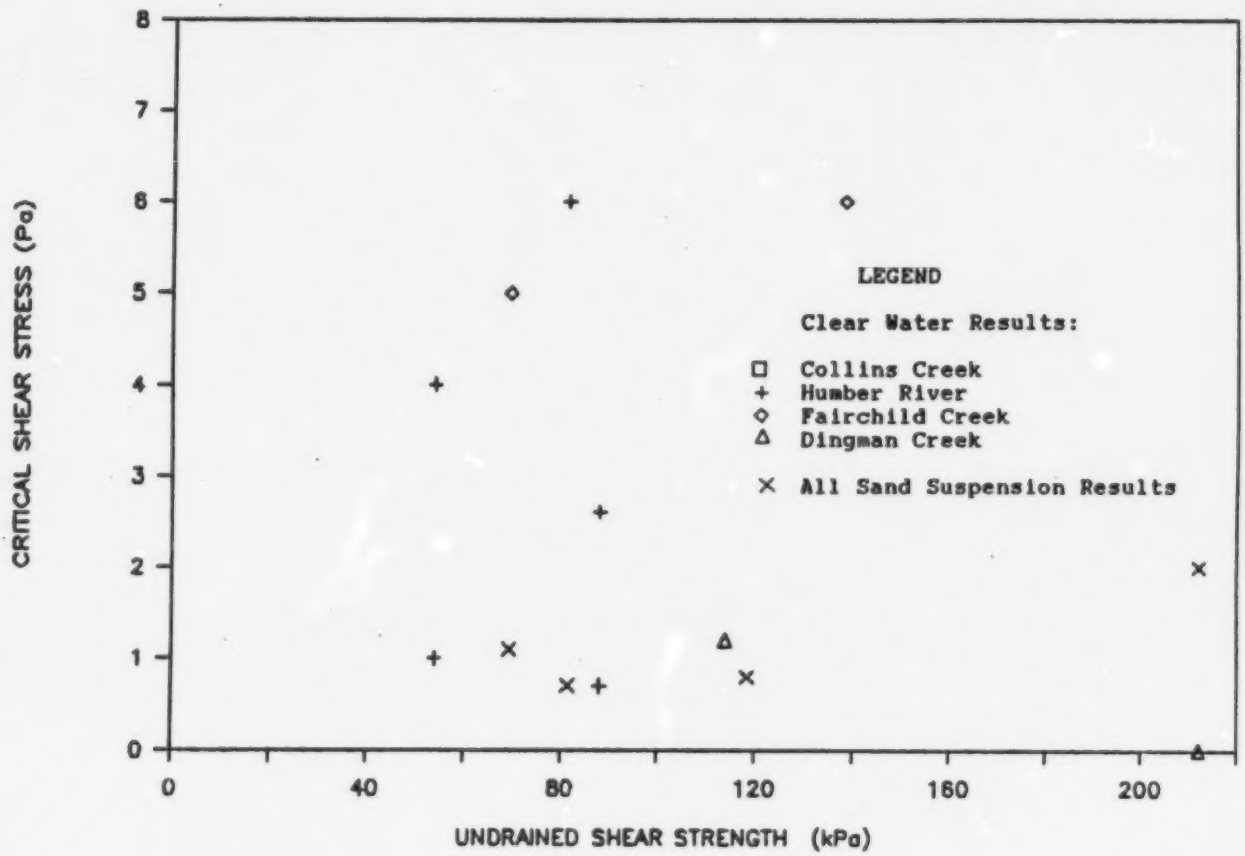


Figure 27/ Clear Water Average Values

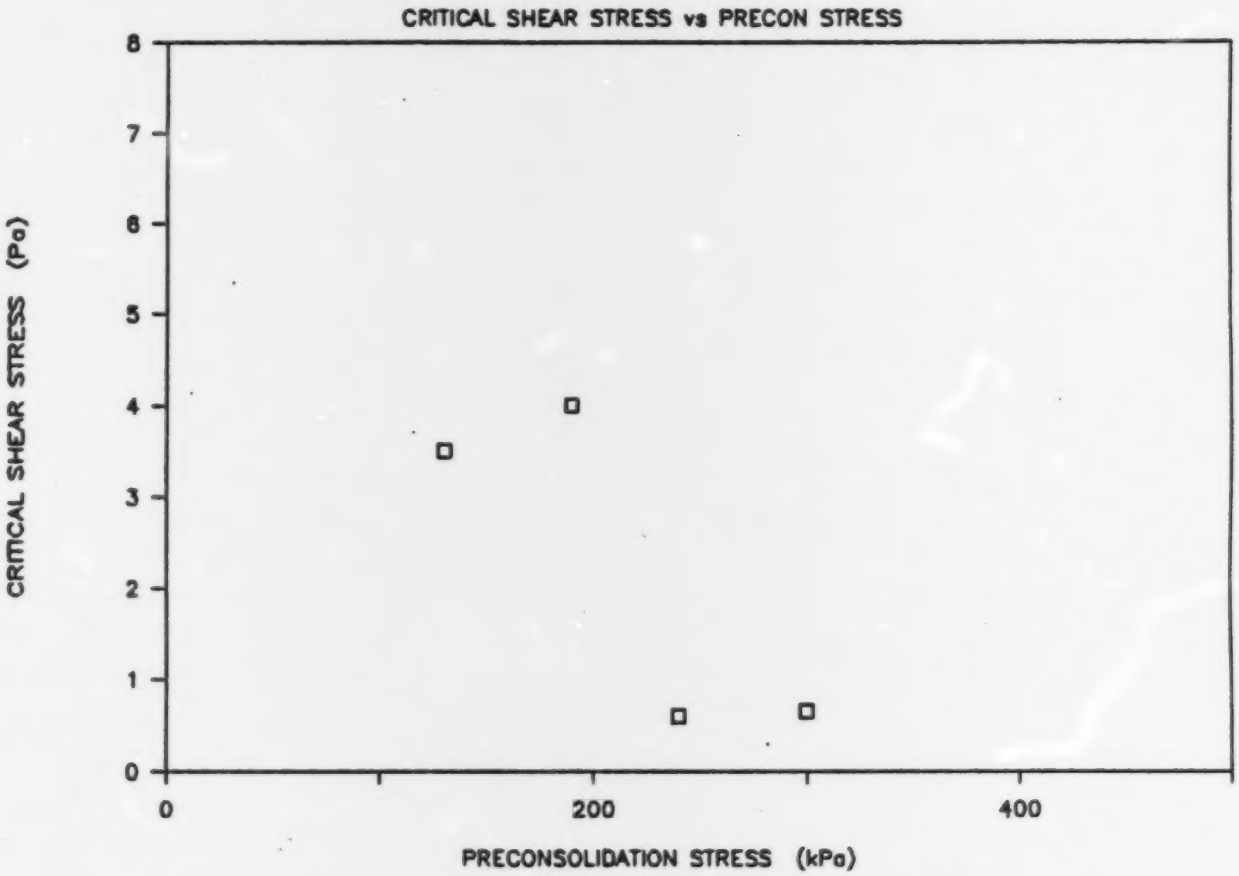


Figure 28/ Erosion-Shear Slope vs Original Water Content

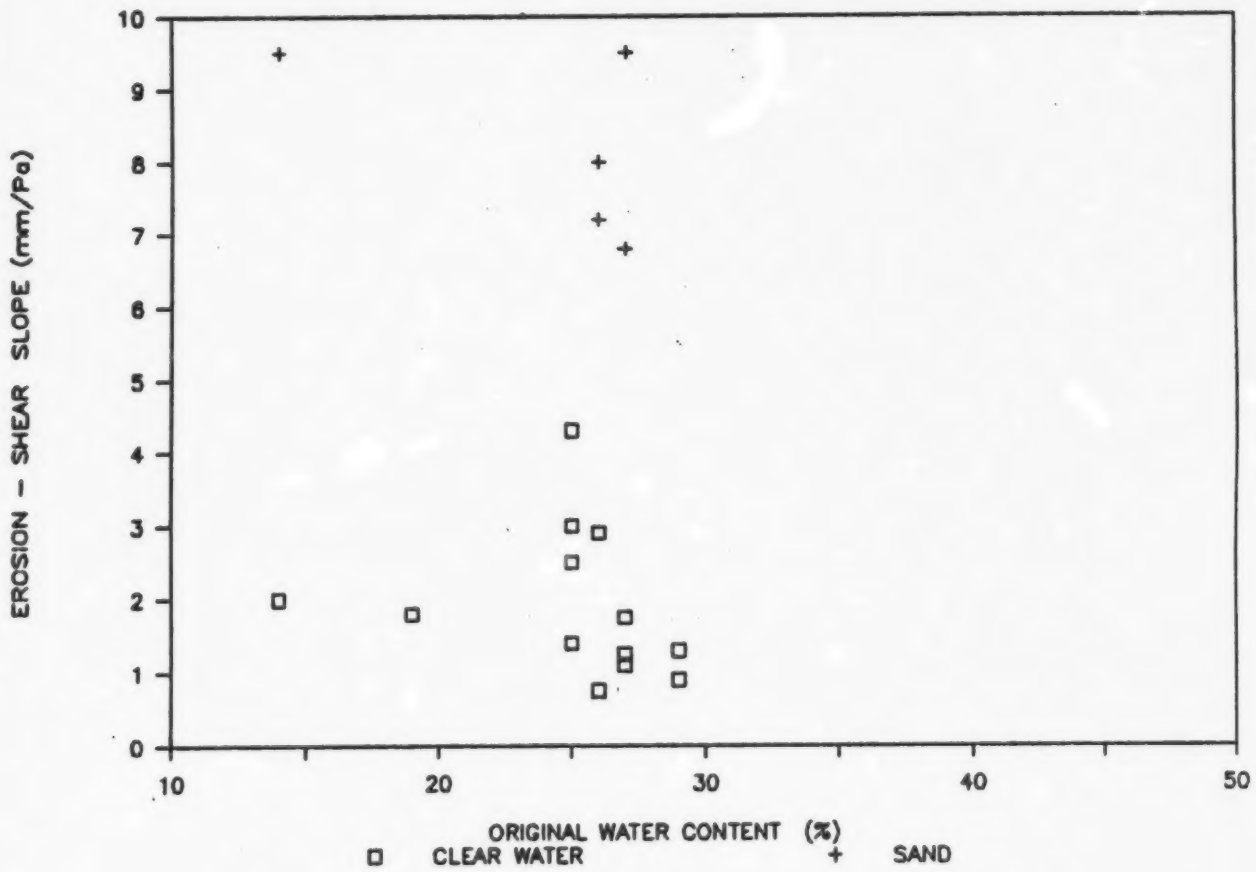


Figure 29/ Erosion-Shear Slope vs Final Water Content

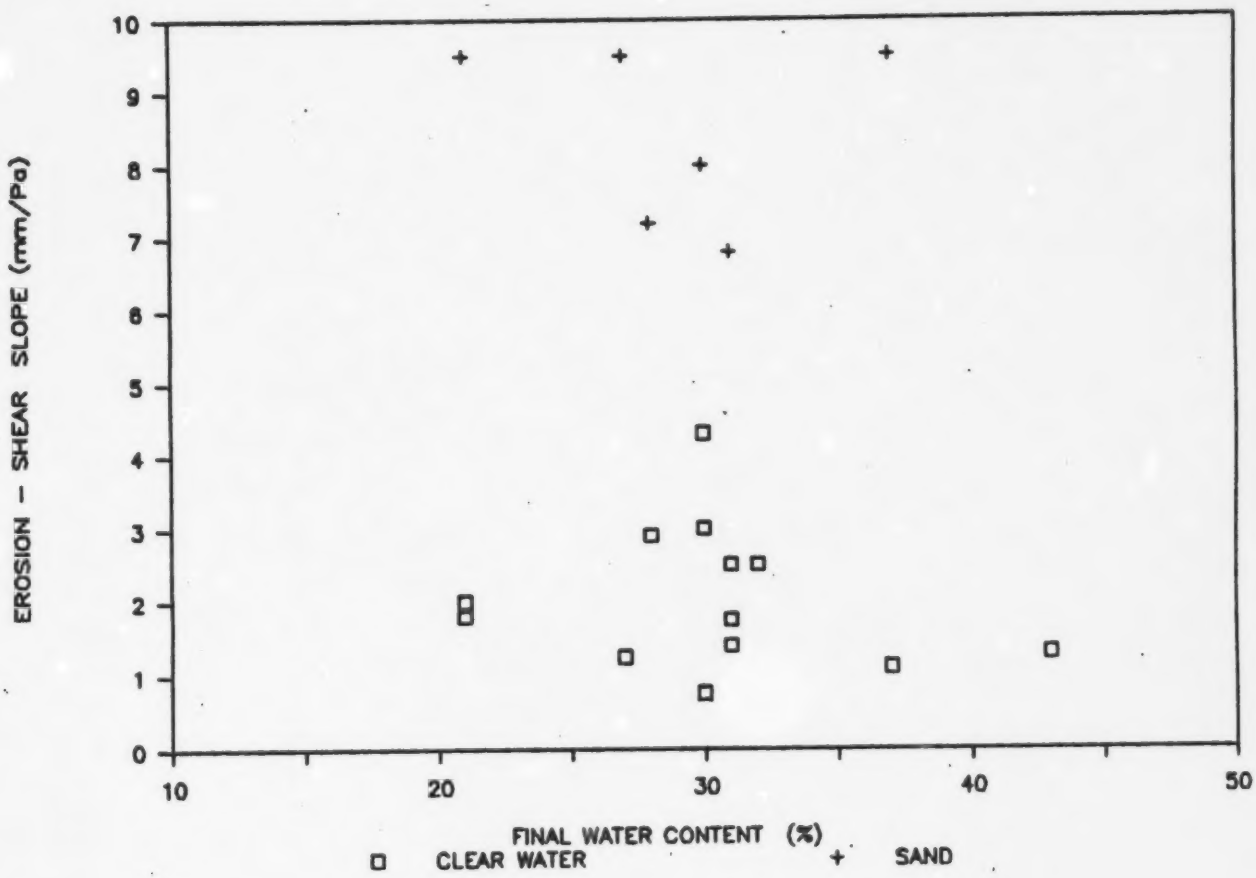


Figure 30/ Erosion-Shear Slope vs Clay Content

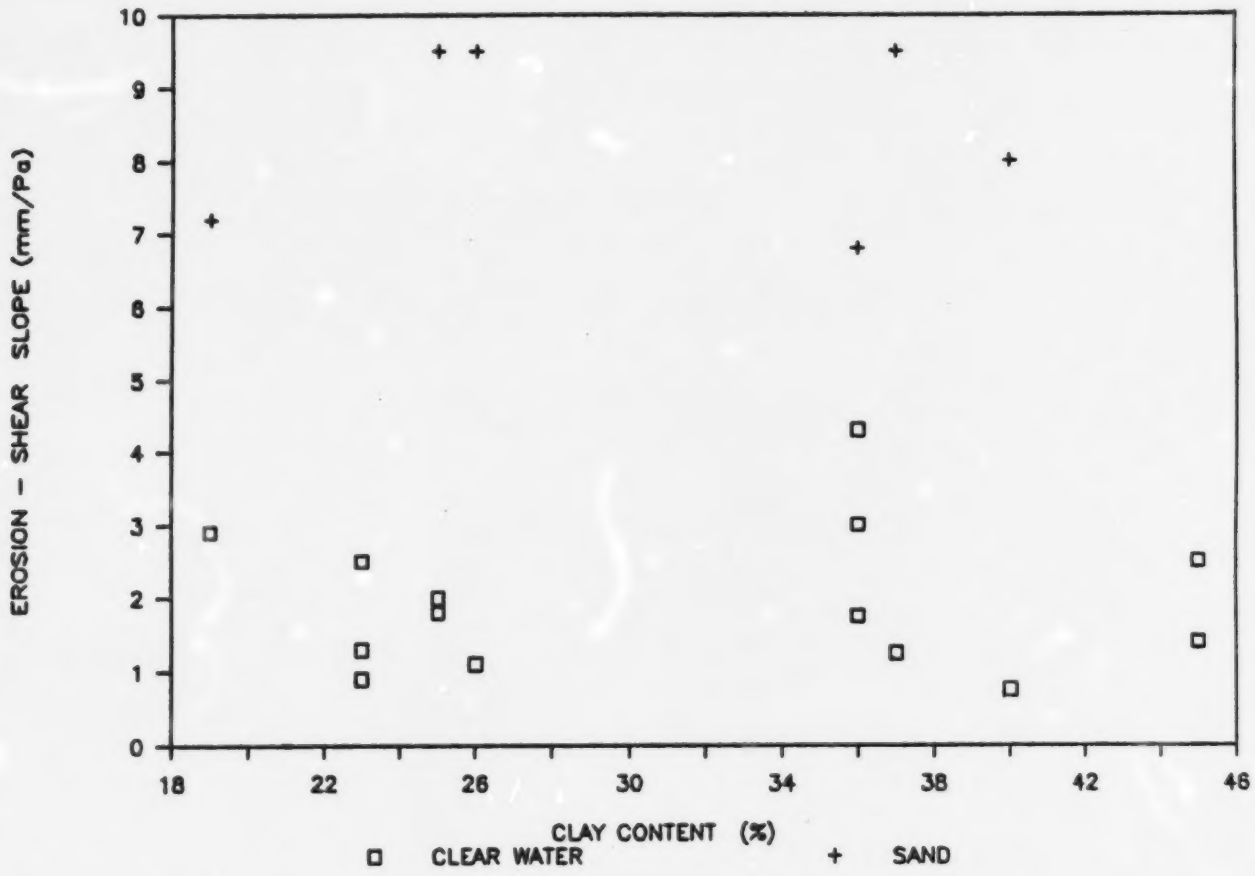


Figure 31/ Erosion-Shear Slope vs Plasticity Index

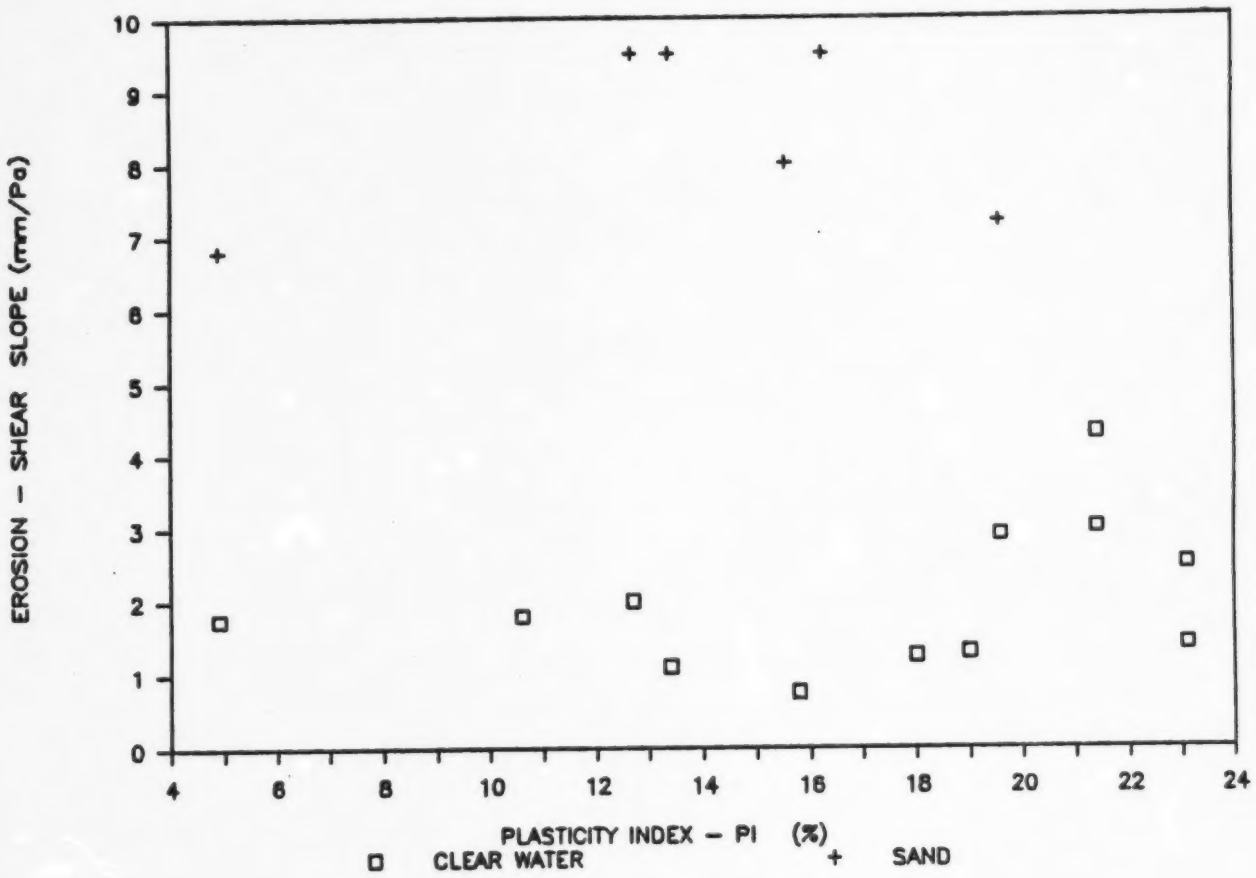


Figure 32/ Erosion-Shear Slope vs Precon Stress

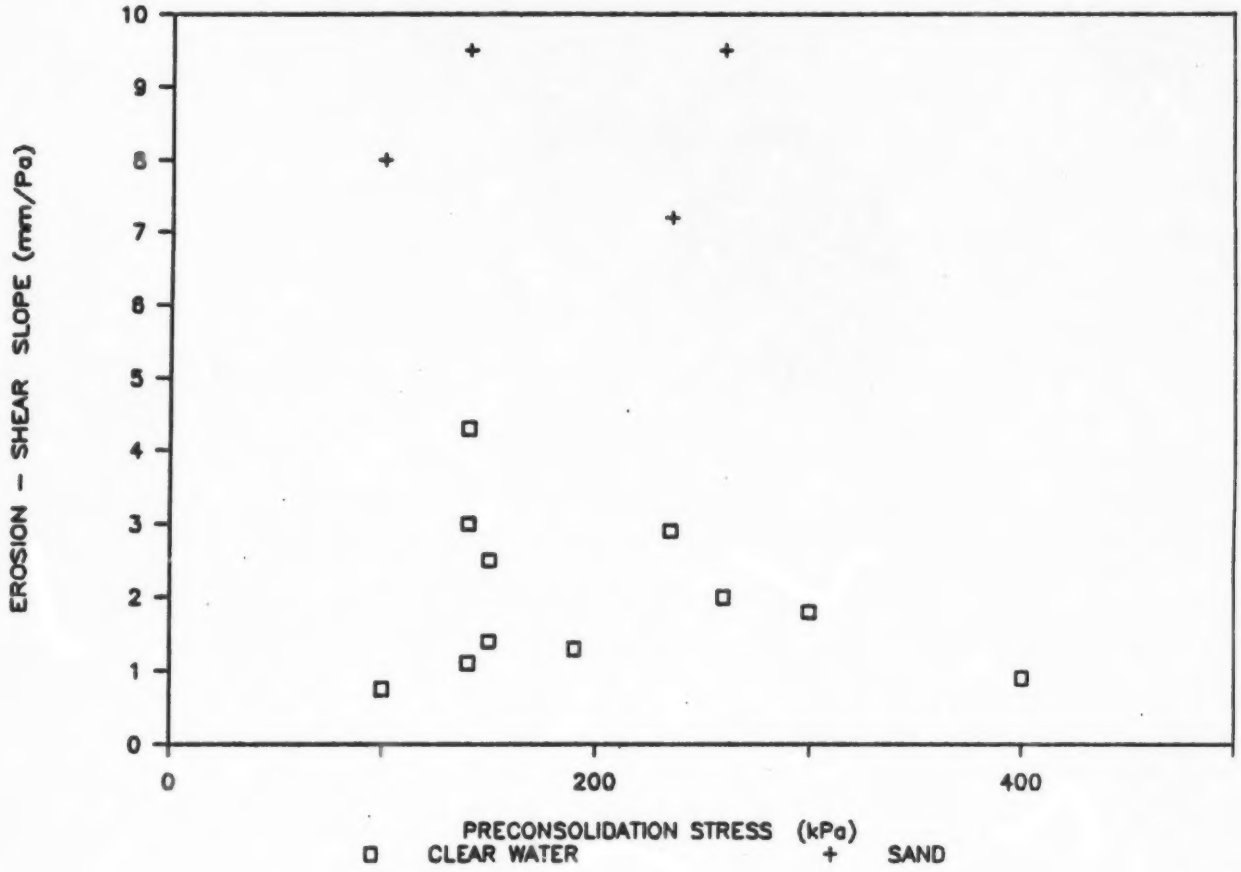


Figure 33/ Erosion-Shear Slope vs Shear Strength

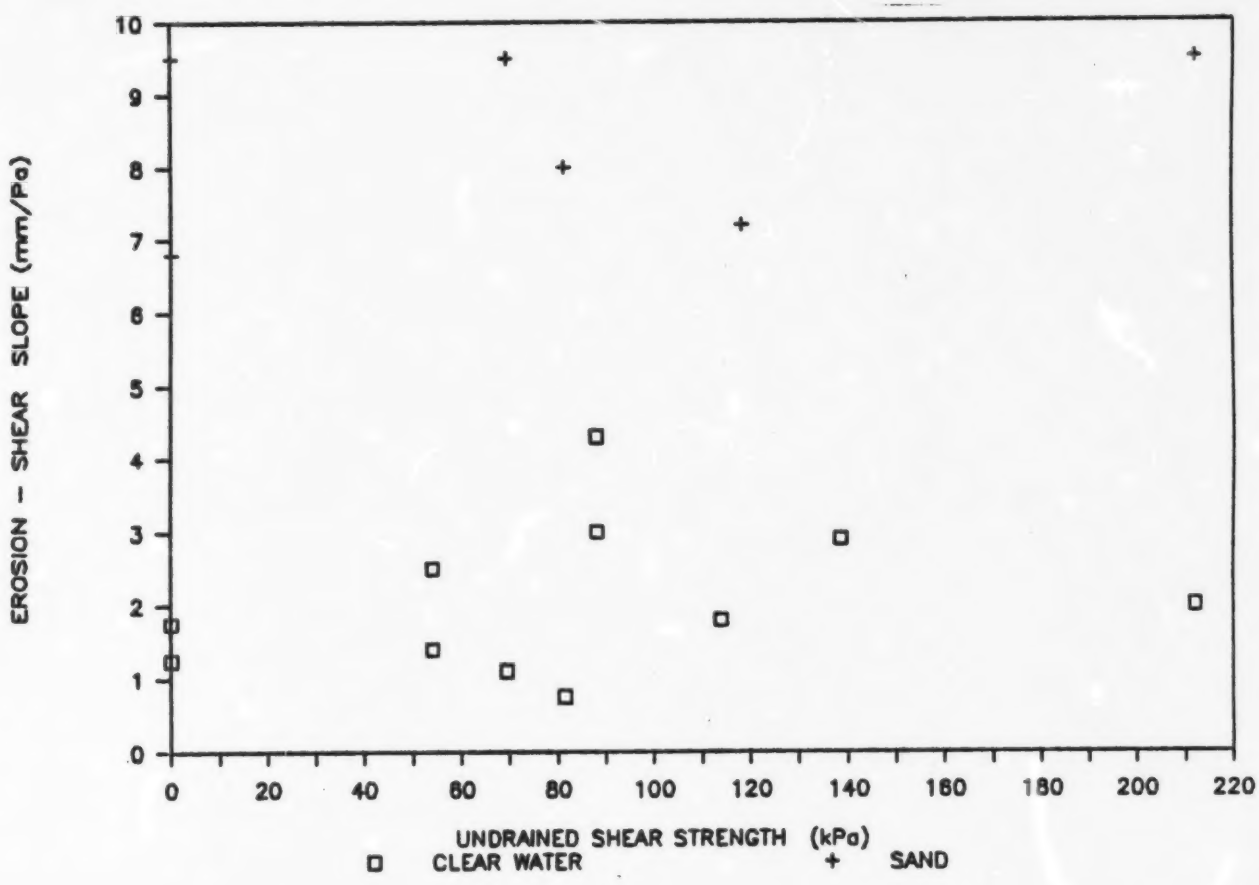


Figure 34/ Clear Water Average Values

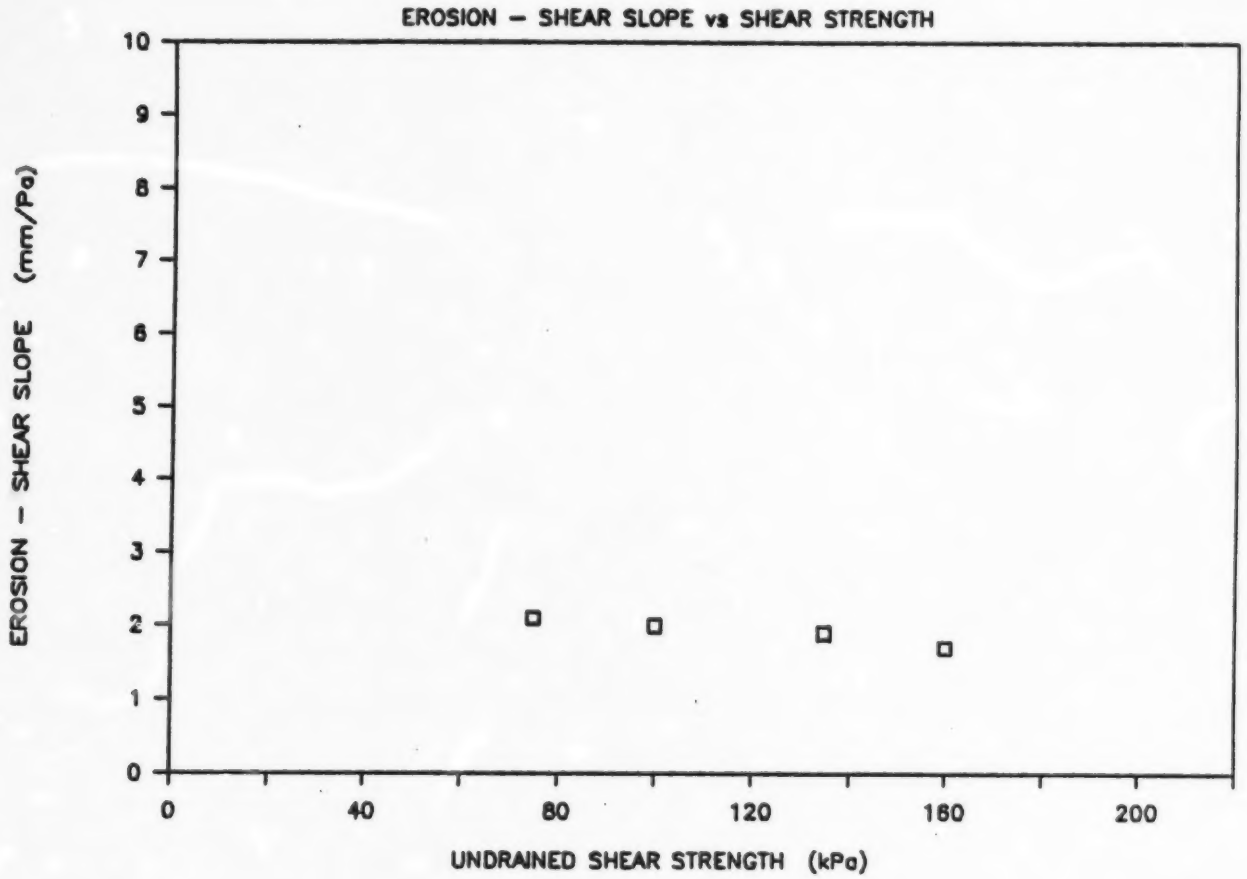
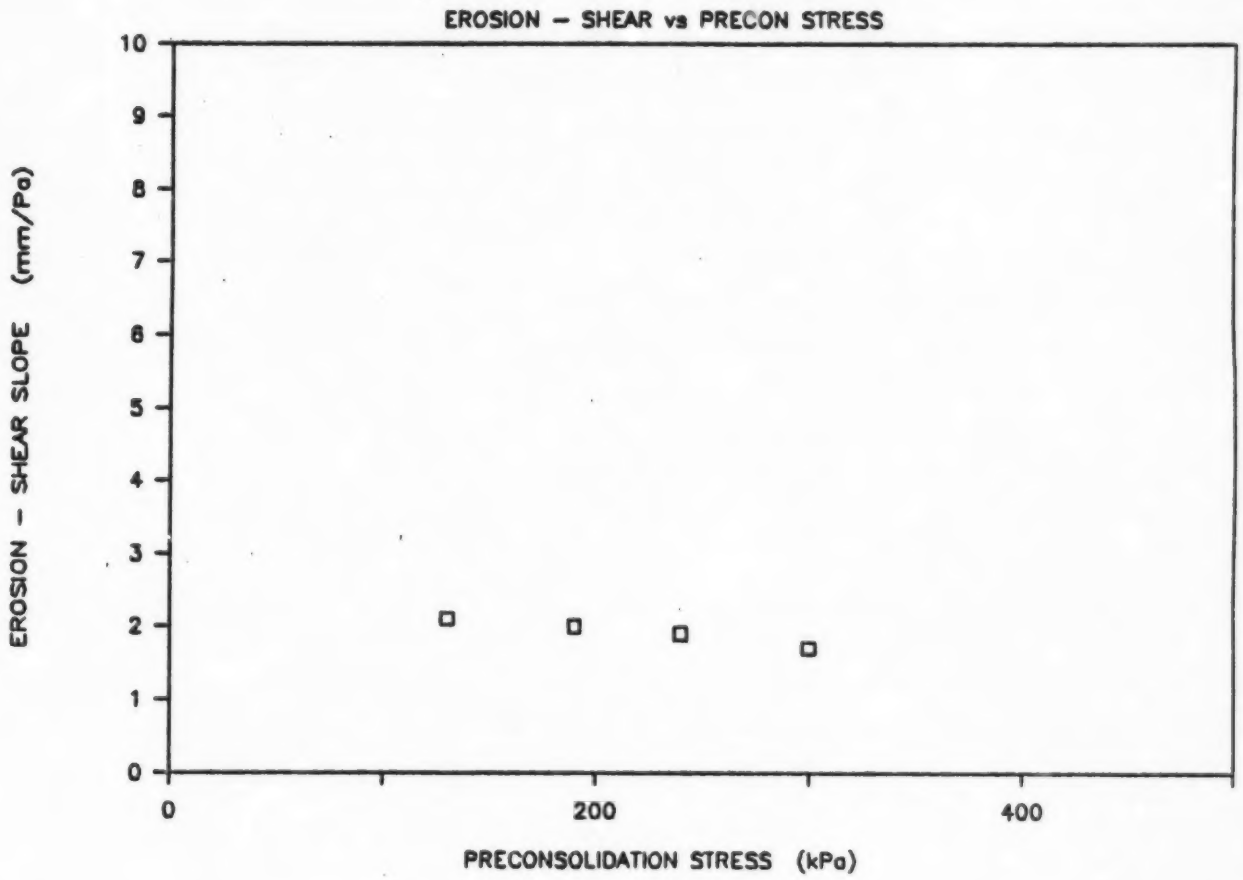


Figure 35/ Clear Water Average Values



Appendix 1/ Collins Creek

The Collins Creek site is located at the crossing of Highway 401 over Collins Creek, to the west of Highway 38 near Kingston, Ontario. The clay, known as the Napanee lacustrine clay (Gillespie et al., 1966), is a soft, grey clay with silt varves (Hunting Technical and Exploration Services Limited, 1958), and is exposed in areas along the margin of the stream as well as buried by up to 0.6 m of organic mucks and marsh grasses. The upper one to two metres of this 10 m thick glacio-lacustrine clay have been desiccated or otherwise altered to have a higher shearing strength than the underlying unaltered clay (Hunting Technical and Exploration Services Limited, 1958). The preconsolidation stress of the clay buried beneath the organic mud was found to be about 190 kPa, while for the exposed clays it was found to be 460 kPa.

The soil samples were found to be heavily fractured, both perpendicular and parallel to the original, horizontal bedding plane surfaces, possibly due to both desiccation and post-depositional unloading. To minimize the possible effects of any seasonal drying, the samples were collected from the bank of the stream, at or below the mean water level. The water content of these samples varies from 29 to 38.8%. The procedure for the flume testing, the results of which are presented below, is outlined in Chapter 6 of this report.

The erosion followed a predictable pattern in all of the Collins Creek samples. At the velocities used in the tests, which generated shear stress values ranging from 0.52 to 2.6 Pa, erosion took place by either flaking or spalling of large particles bounded by the fractures. Once the critical shear stress for erosion was reached, i.e., the shear stress under which erosion begins, an initially rapid rate of erosion was noted which gradually tapered off with time. By the end of the two-hour test interval, little additional erosion occurred. Any subsequent increase in velocity re-initiated the erosion process. The initial tests were run at low velocities, yet had relatively rapid erosion due to the presence of large numbers of small fractures in the upper 10 mm of the sample. At velocities equal to or in excess of 1 m/s larger sets of fractures, typically not evident on the sample surface, began to control the erosion process and, as a result, the size of the particles spalling off the sample surface increased. The increases in the velocity also produced small amounts of erosion by the flaking of small particles from the irregular surface of the sample. A secondary effect was noticed in some of the samples. The presence of rootlets from the overlying marsh grasses added a random reinforcement to the soil as the rootlets held clay particles in place that otherwise would have been eroded by the current. The presence of the rootlets was not homogeneous throughout the sample and in some cases, the roots were decaying and had little remaining strength.

The top of Sample 4 was tested with sand included in the water to initiate scour. The low shear stresses and high erosion rates in the clear water tests made it necessary to begin the sand tests at very low water velocities in the order of 0.5 m/s. At these low velocities, the sand moved as a discontinuous rolling bedload causing only minor abrasion of protruding inhomogeneities in the soil surface. Fractures in the sample surface became sites of preferential erosion, deepening slightly, filling with sand and rounding the otherwise angular edges. Saltating sand grains caused some surface pitting of the sample. The water velocity was increased after each two-hour test to monitor the change in the rates and style of erosion. As the water velocity increased, large particles would spall off in the same manner as erosion in clear water. As soon as the large hollows appeared, the bedload sand would fill in the hollows. As a result, the surface which was essentially eroded by clear water was actually buried by the mobile sand layer, protecting the sample surface from further erosive action. When the flume was turned off, the sand stopped moving and collected in the hollows. This sand layer was removed to measure the actual eroded surface.

Appendix 2/ Humber River

The Humber River site is located on the cutbank of a meander loop on the Humber River in the Towntree Mills Park at the corner of Islington and Finch Avenues in Toronto. The clayey soils at this site overlie a crossbedded sands and silts unit and, in turn, are overlain by horizontally bedded to crossbedded silts, sands and gravels. At the time of sampling, the mean water level of the river was approximately 1 m below the lowest occurrence of the clay horizon. The total vertical thickness of the clay at this location is approximately 4 m. A total of five blocks of soil, four of which were tested, were removed from the site.

The clay is a firm, grey, silty clay with occasional silt and sand lenses as well as scattered gravels. The Unified Classification for the samples is silty clay to clayey silt, falling in the CL class. Preconsolidation stresses for Samples 1, 2 and 3 were found to be 140, 100 and 110 kPa, respectively. Some fracturing was evident on the surfaces cleared for sampling and in some cases these fractures appeared to be quite extensive. The sampling locations were chosen in an attempt to minimize the amount of fracturing present in the sample as well as to represent the soils from different parts of the clay stratum.

The erosion tests of these samples followed the procedure described in Chapter 6. Erosion of the Humber River samples is controlled in part by the presence of fractures or bedding within the sample. The fractures tended to be widely spaced in these soils and, as a result, the majority of any fracture-bounded particles were relatively large, often with lengths and widths in excess of 10 to 20 mm and thicknesses of up to 10 mm. Because of the large size of these particles, they had a tendency to break away only at the higher velocities that occurred during the tests. In addition to the erosion of fracture-bounded particles, the soils also eroded by the flaking and peeling of small, thin layers of clay. Visual examination of the soil samples did not indicate the presence of thin laminations or bedding, thus these thin layers were not controlled by any obvious original depositional effects. The absorption of water by the soil affected the sample in two ways; first, the water softened a thin layer of the soil which was subsequently more easily eroded and, second, the addition of the water caused the surface to expand which in some cases caused a decrease in the measured volume of material eroded.

The standard pattern of erosion at almost all of the velocities tested was to have larger particles either spall or peel away leaving a local inhomogeneity in the sample surface. Smaller particles then flaked and peeled away from the margins of these hollows until some state of equilibrium was reached and no further erosion occurred until further larger particles once again eroded away. If no

large particles eroded from the sample surface, this surface appeared to remain untouched, with the exception of very small volumes of material flaking away. At the highest velocities used with the Humber River samples (2.9 to 3.0 m/s), small particles smeared off the otherwise stable sample surfaces. Over the course of the test period, this sample surface would smooth out and develop a faint, flow-parallel streakiness. Erosion by the spalling and breaking of larger pieces continued at these high velocities. Examination of the plot of eroded depth vs shear stress for the clear water tests shows significant scatter around a general trend. This scatter is attributed to the variability between the different samples such as the amount of fracturing and the presence of silt lenses. For example, the upper surface of Sample 1 contained a large silt lens at the downstream end of the sample. The initiation of a low velocity flow caused rapid erosion of this lens, yielding a very high measurement of eroded volume at a relatively low shear stress.

Erosion tests with the Humber River samples for a sand/water mixture were conducted at velocities ranging from 0.65 to 1.2 m/s. At these velocities, the sand moved as a rolling and saltating, discontinuous bedload. While a wide distribution of the eroded depths for the clear water tests for Samples 1, 2 and 5 is evident, the same samples when tested with the sand/water mixture all have remarkably similar eroded depths at similar shear stresses. This is interpreted to show that the presence of the sand as an abrading agent plays the major role in the erosion of the samples. This contrasts with Collins Creek Sample 4 where a significant amount of fracture-controlled erosion occurred at velocities below those required to initiate the motion of the 2 mm sand. The tests showed that the presence of the sand in the water resulted in general abrasion across the surface of the sample and generally formed some flow-parallel gullies and irregularly-shaped bumps and hollows. Spalling and flaking of the sample surface did not occur during the sand tests.

Appendix 3/ Fairchild Creek

Fairchild Creek is an incised, meandering stream located in the Brantford area of southern Ontario. The site sampled is located at the proposed bridge crossing of the extension of Highway 403 on property currently owned by Greenlife Proteins Limited. Access to the sample site was from Johnson Road.

The local stratigraphy consists of 4 m of loose silt with traces of organic material overlying 9.6 m of alternating layers of firm to stiff clayey silt and silty clay and which, in turn, overlies a firm clayey silt with some sand and gravel (Johnson, 1979). The samples were removed from the upper part of the alternating clayey silt and silty clay layers. The samples were collected at the position of mean water level at the time of sampling (October, 1987) and approximately 1 to 1.5 m below the boundary with the overlying loose silt. A total of four samples were collected at this site, three of which have been tested.

The samples tested are firm, brown, clayey silts with a Unified Classification of CL to ML or OL. Preconsolidation stresses for Samples 3 and 4 were found to be 200 and 280 kPa, respectively. By way of comparison, the preconsolidation stress of a sample taken from a borehole on the opposite bank of the stream at a depth of 6 m below borehole top was found to be 498 kPa (Johnson, 1979). This would correspond to a depth approximately 4 to 5 m below the position sampled for this study. No fractures were visible on the surfaces cleared for sampling and no pebbles were encountered, as was the case for the Humber River and Dingman Creek samples. Some sand lenses were evident in the sample areas, though the samples were carefully taken to avoid sampling the sandier areas.

The flume tests of the soil samples were conducted at velocities ranging from 0.65 to 2.29 m/s for the clear water tests and at velocities of 0.59 to 1.19 m/s for the sand tests. Testing revealed fractures in all three of the tested samples, and both bedding planes and fractures in Sample 4. These features were not evident in the field. The fractures that appeared as the soil began to erode were very small and localized. As with the samples for the other sites, however, these fractures were the locations of preferential erosion due to the locally weaker nature of the soils there. The soil eroded by spalling of fracture-bounded chunks, particularly at the higher test velocities, followed by the flaking of small particles from the margins of the newly-exposed surfaces. In areas where there were no fractures present, the soils eroded by the flaking away of small, irregular layers, commonly only millimetres in length and width. Immersion of the sample in the flume had little

effect on the water content of the top of the sample and very little expansion of the upper layer(s) was noticed.

The thin bedding in the soil, particularly evident in Sample 4, could not be subsampled for detailed grain size analysis but appeared to consist of alternating layers of silty clay and clayey silt with thicknesses of 3 to 5 mm. The erosion of Sample 4 was distinctly controlled by the presence of these bedding planes with relatively large (up to 20 mm long) tabular fragments breaking away at the higher velocities. In all cases, both in the presence and absence of bedding and/or fractures, the surface distribution of the silts and clays played a significant role in the location and rate of erosion. The siltier areas showed rapid preferential erosion both by the scour of the silt grains and the flaking away of small fragments.

With the addition of sand to the water, the erosion became very similar to that of the other soil types tested. The soil, which could be classified as firm, eroded rapidly with the presence of the sand. Comparison with the other soil types showed that similar erosion depths occurred for equivalent shear stresses, highlighting the controlling influence of the eroding sediment on the rate of erosion. In addition to controlling the rate of erosion, the presence of sand also appeared to inhibit the erosion of the soil by the flaking or spalling that occured during the clear water tests.

Appendix 4/ Dingman Creek

The fourth site selected for testing was the intersection of Dingman Creek with Victoria Road, Concession III in the Township of Westminster, near London, Ontario. This site, located adjacent to an existing bridge crossing, required some excavation to allow sampling of the grey, clayey silty till. This till was overlain by a thin sand and gravel layer and by surface topsoil It was underlain by a brown, silty clay (Golder Associates, 1987). Examination revealed the presence of two distinct layers within the till. The upper layer, with a thickness of approximately 0.5 m, is a clayey silt with no gravels. The lower layer is a clayey silty till with abundant gravels throughout the soil. Samples 1, 2 and 3 were taken from the lower clayey silty till from positions at or below the present stream bed. Samples 4 and 5 were collected from the upper clayey silt at a height approximately 1 m above the stream bed. Four of these samples were tested.

Despite the care taken when collecting the samples, it was very difficult to collect any soil samples that did not contain any gravels. As a result, the soils showed a wide variation in textural composition. Sample 2 for example, is a clayey silt from the same stratigraphic horizon as the till-like Samples 1 and 3. The presence of the gravels had an effect on both the sampling of the soils and on the flume test results. Due to the nature of the sampling technique, any stones that were encountered by the sides of the sample box were shifted, resulting in an artificial fracturing of the soil. This fracturing played a significant role in controlling the erosion of both Samples 1 and 3. The preconsolidation stresses for Samples 1, 2 and 3 were found by this study to be 280, 350 and 250 kPa, respectively. The consolidation test performed on Sample 4 failed to record the preconsolidation pressure. Immersion of the samples in the flume caused noticeable expansion of the top of the sample and associated softening of the surface in contact with the water.

Clear water tests with the till-like samples (Samples 1, 3) produced rapid erosion rates and corresponding high erosion volumes at relatively low shear stresses. Samples 1 and 3 were affected by the fracturing induced by the movement of stones during the field sampling and the preparation of the samples in the laboratory. This, in part, accounts for the erosion by the spalling off of fracture-bounded particles of a wide variety of sizes, ranging from small, tabular flakes to large blocky chunks. While much of the erosion occurred in the vicinity of the stonier areas of the sample, additional erosion took place on the remainder of the sample surface both by the spalling of blocky chunks and the flaking of small pieces. Of the two samples, Sample 3 was more affected by the fracturing due to the presence of stones than was Sample 1. The very rapid erosion rate of Sample 4 with clear water was found to be due to the existence of very thin beds within the clay till. The clay eroded by the removal of large numbers of small, oblong flakes, generally up to 5 mm long.